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US Army Corps of Engineers®

San Clemente Shoreline Feasibility Study
Orange County, California

Coastal Engineering Appendix

Los Angeles District
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1 GENERAL

1.1 Introduction

1. This appendix describes technical design aspects for the San Clemente Shoreline Feasibility Study for shoreline improvements in the City of San Clemente, California. The purpose of this appendix is to conduct a detailed evaluation of the existing conditions and of the proposed alternatives for determination of the recommended plan. Evaluation of these alternatives includes an analysis of baseline coastal processes, detailed technical design, and potential design impacts on shoreline processes.

2. This technical appendix references significantly a predecessor document, the Coast of California Storm and Tidal Wave Study (CCSTWS) prepared by the Los Angeles District (USACE-LAD, 1991).

3. This study at San Clemente is conducted under the SI (metric) system of measurement as part of the U.S. Army Corps of Engineers compliance with Federal policy that requires all new Federally funded construction work be conducted in SI units. Much of the existing coastal processes analyses were originally developed utilizing the Imperial measurement system (feet). All units and/or dimensions are converted to metric equivalents using standard conversion factors. Some dimensions are shown in both systems (Imperial units in parentheses) to aid in understanding comparisons with historical documents.

1.2 Risk and Uncertainty Analysis

4. This study at San Clemente is conducted using the principles of Risk & Uncertainty (R/U) in compliance with U.S. Army Corps of Engineers policy that requires all new Federally funded flood control and flood damage reduction studies incorporate the principles of R/U. U.S. Army Corps of Engineers policy guidelines for R/U are defined in Engineer Regulation 1105-2-101 (Dept. of Army 1996). R/U is intrinsic in water resources planning and design and arises from measurement errors and the inherent variability of complex physical phenomena.

5. At the time of this study there is no Corps of Engineers standard model for R/U analysis for coastal zone storm damage reduction studies. A nationwide model for coastal zone R/U analysis is currently under development by the Corps of Engineers, however, the model completion schedule did not allow for utilization for this study. Other existing Corps of Engineers coastal zone R/U models were evaluated for implementation under this study. These R/U models included Generalized Risk AND Uncertainty Coastal model (GRANDUC) developed by the Wilmington District and the Risk Storm Damage Model developed by the Jacksonville District. These R/U models, which were developed independently at different times by the respective districts, were evaluated extensively. Due to a variety of engineering considerations, technical, programming language, and other reasons, each model was rejected for use for this study. An interim model was developed that is uniquely applicable to the physics and storm conditions that are experienced within the Los Angeles District. This model utilizes many of the concepts and to a large degree is consistent with the nationwide model under development.

6. This model incorporates R/U by utilizing probability distributions for variables and design parameters where appropriate. It is recognized that the "true" values of the design variables and parameters are frequently not known with certainty and can take a range of values. However, the likelihood of a parameter taking on a particular value by a probability distribution can be described. The probability distribution may be described by its own parameters such as mean, standard deviation, shape, and scale. In some cases, the probability distribution for a parameter may be well established in the engineering literature, or in other cases a best-fit distribution of the measured data may be applied.

7. The principles of Monte Carlo Simulation are used as the numerical integration technique. The proprietary computer program @RISK (Palisade, 2002) was used to run the R/U analysis. @RISK is an add-in to a standard industry spreadsheet package that provides the necessary tools for executing a Monte Carlo Simulation.

1.3 Previous Reports

8. There are no previous Federal reports that directly address the San Clemente study area being considered under this authority. Some previous reports exist which incorporate the San Clemente area indirectly as a component of the larger coastal region, such as the aforementioned CCSTWS. The prevailing literature typically emphasizes San Juan Creek and the immediate adjacent coastal area encompassed by Doheny State Beach, or the San Onofre area.

2 BASELINE COASTAL AND OCEANOGRAPHIC CONDITIONS

2.1 Study Area Definition

9. The Oceanside Littoral cell is approximately 86 km (53.5 miles) long extending from Dana Point to the La Jolla submarine canyon. Dana Point, the north end of the cell at Km 133.6 (Mile 83) is a near complete barrier to the littoral transport of sand. Point La Jolla at Km 47.5 (Mile 29.5), the south end of the cell, is also a near-complete barrier. Analysis of shoreline changes in the Oceanside Cell addresses six sub-reaches within the cell. The sub-reach inclusive of the study area is the San Mateo – Dana Point cell between Km 117.5-133.6 (Mile 73-83). The Oceanside Littoral cell boundaries were selected based on geomorphic and cultural features such as river and lagoon entrances, harbor structures, and other shoreline change features.

10. The current study area is bounded between San Mateo Point and Dana Point. The study area is approximately 12.1 km (7.5 miles) long. The study area is further sub-divided into ten reaches for reference purposes numbered consecutively Reach 1 – Reach 10. The study area is shown in Figure 1.

11. The Southern California Regional Railroad Authority (SCRRA) railroad line bisects the length of the project area. The railroad is constructed on conventional elevated crushed rock ballast. The railroad is constructed along the base of the bluff throughout the reach and is a prominent feature that completely separates the active beach from the bluff. The beach is only accessible via pedestrian underpasses/overpasses or storm water culverts interspersed along the length. Whereas typically the beach and bluffs are considered joined together as components in an active littoral system, the presence of the railroad completely isolates the bluff from the beach. Virtually all sediment inputs and bluff-related influences are restricted. Furthermore, the railroad resembles and acts as a protective revetment. In several areas the railroad is the primary structural feature that protects the bluffs and upland development from direct wave attack. In several areas the beach has completely eroded and exposed the railroad, thus, the railroad has fixed the position of the shoreline. Thus the railroad is the de facto protective structure. The railroad, acting as a protective structure and fixing the position of the shoreline, is assumed to be permanent and always exist throughout the project lifecycle. Therefore, the railroad is considered the landward boundary and no storm damages are considered landward of the railroad.

12. SCRRA has incorporated discontinuous levels of protective rock armoring along the railroad length. Four of the reaches consist of conventional rock ballast gradation which ranges from ¾” – 2-1/2”. Four of the reaches have been improved with rock armoring of approximately 100 lbs – 4 tons (~1 ft – 4 ft) on the seaward slope. The general condition of the revetment is not uniform and appears to be fair/poor along the entire length. Information obtained from SCRRA indicates that the revetment armor stone was not individually placed in the same manner in which conventional coastal armor stone structures are constructed. Instead the revetment was constructed via rail car side dump methods. However, even though the design and methods of construction were inconsistent with conventional coastal engineering practice, the larger armor stone affords substantially greater protection against hydraulic forces over conventionally sized rock ballast and offer substantial benefits to the railroad structure. This improvement in protective armoring is a primary consideration for project reach delineation.

13. The reach boundaries are defined in meters northward from San Mateo Point and shoreline features located within each reach are described. The project baseline is assumed to be the seaward rail of the SCRRA railroad. Since the railroad is assumed to be a constant feature throughout the project lifecycle,

the seaward rail provided a convenient feature to define a horizontal alignment. The project reaches are defined based on an arbitrary assumed Station 0+000 at San Mateo Point.

14. Reach 1: Reach 1 extends from San Mateo Point at Sta 0+146 to Sta 1+115. The reach is 969 m long and is the southern portion of San Clemente State Beach. The beach width is zero at the southern boundary and gradually increases to 41 m wide. The railroad track elevation is approximately +6.4 m[1]. The railroad seaward slope incorporates the improved armor stone protection, has a slope of 1H:1V, and a crest elevation of approximately +7.0 m. There are no structures seaward of the railroad; some residential structures exist immediately landward of the railroad.

15. Reach 2: Reach 2 extends from Sta 1+115 to Sta 1+795. The reach is 680 m long and is encompassed within San Clemente State Beach. The beach width is approximately 40 m wide at the southern boundary and gradually decreases to 9 m wide. The railroad track elevation is approximately +6.4-7.6 m. The railroad seaward slope incorporates the conventional ballast construction, has a slope of 1H:1V, and a crest elevation of approximately +6.4-7.6 m. There are no structures landward of the railroad; the underpass for San Clemente State Beach is included within this reach.

16. Reach 3: Reach 3extends from Sta 1+795 to Sta 2+395. The reach is 600 m long is encompassed within San Clemente State Beach. The beach width is approximately 9 m wide at the southern boundary and quickly becomes zero throughout the remainder of the reach. The railroad track elevation is approximately +6.4 m. The railroad seaward slope incorporates the improved armor stone protection, has a slope of 1H:1V, and a crest elevation of approximately +7.0 m. There are no structures landward of the railroad; Calafia Beach Park is on the landward side of the railroad.

17. Reach 4: Reach 4 extends from Sta 2+395 to Sta 3+127. The reach is 732 m long and encompasses San Clemente State Beach on the southern 30% and City of San Clemente on the northern 70%. The beach width is approximately 30 m wide at the southern boundary, transitions to 60 m wide in the middle, and transitions to 10 m wide at the northern boundary. The railroad track elevation is approximately +6.3 m. The railroad seaward slope incorporates the conventional ballast construction, has a slope of 1H:1V, and a crest elevation of approximately +6.3 m. There are no structures landward of the railroad; some residential structures exist immediately landward of the railroad.

18. Reach 5: Reach 5 extends from Sta 3+127 to Sta 3+540. The reach is 413 long and is encompassed within the City of San Clemente. The beach width is 0 m wide throughout the reach. The railroad track elevation is approximately +6.5 m. The railroad seaward slope incorporates the improved armor stone protection, has a slope of 1H:1V, and a crest elevation of approximately +6.5 m. There are no structures landward of the railroad; some residential structures exist immediately landward of the railroad.

19. Reach 6: Reach 6 extends from Sta 3+540 to Sta 4+580. The reach is 1040 m long and is encompassed within the City of San Clemente. The beach width meanders from 0 m wide to 23 m to 0 m to 39 m and back to 0 m along the reach. The railroad track elevation is approximately +6.3 m. The railroad seaward slope incorporates the conventional ballast construction, has a slope of 1H:1V, and a crest elevation of approximately +6.9 m. This reach includes the majority of the significant structures along the beach.

20. Reach 7: Reach 7 extends from Sta 4+580 to Sta 5+661. The reach is 1081 m long and is encompassed within the City of San Clemente and is known as “Mariposa Point”. The beach width is 0 m wide throughout the reach. The railroad track elevation is approximately +6.5 m. The railroad seaward slope incorporates the improved armor stone protection, has a slope of 1H:1V, and a crest elevation of approximately +6.9 m. Historical information indicates that this reach has been armored with revetment since at least the 1930’s. There are no structures seaward or landward of the railroad.

21. Reach 8: Reach 8 extends from Sta 5+661 to Sta 6+008. The reach is 347 m long and is encompassed within the City of San Clemente. The beach width varies from 40 m wide at the southern

[1] All elevations and/or depths are in meters referred to Mean Lower Low Water (MLLW) unless otherwise noted.

boundary to 0 m at the northern boundary. The railroad track elevation is approximately +6.3 m. The railroad seaward slope incorporates the conventional ballast construction, has a slope of 1H:1V, and a crest elevation of approximately +6.9 m.

22. Reach 9: Reach 9 extends from Sta 6+008 to Sta 7+109. The reach is 1101 m long and is encompassed within the City of San Clemente and is known as “Capistrano Shores”. Capistrano Shores is a private community of manufactured housing constructed in the 1950’s. A timber seawall that is fronted by a rubblemound rock revetment protects the reach. The armor stone is estimated to be 2-5 tons, has a slope of 1H:1V, and a crest elevation of approximately +6.0 m. The general condition of the revetment is not uniform and appears to be fair/poor along the entire length. The beach width is 0 m wide throughout the reach. The railroad is located substantially landward of the revetment and as such is no longer considered the project landward boundary. There are no structures seaward of the revetment.

23. Reach 10: Reach 10 extends from Sta 7+109 to Dana Point Harbor. The reach is approximately 5,000 m long and extends to the northern boundary of the study area. The Reconnaissance Report identified the potential project area from San Mateo Point to Capistrano Shores (Reach 1-9). The area north of Capistrano Shores is designated Reach 10 and will be considered only in the context of engineering considerations where physical continuity is required and/or necessary for the analysis.

Figure 1 San Clemente Study Area

2.2 Water Levels

2.2.1 Tides

24. Tides at San Clemente are unequal mixed semi-diurnal. Tidal datum elevations at La Jolla, CA are shown in Table 1 and are assumed representative of those at San Clemente. The tidal epoch is 1960 to 1978. La Jolla is a National Ocean Service Primary Tide Station approximately 81 km (50 miles) from San Clemente Pier. Tides typically have spatial characteristics on the order of hundreds of kilometers and the sea level variation between La Jolla and San Clemente is assumed to be negligible.

Table 1 Tidal Datum Elevations at La Jolla, CA

Tidal Datum	Elevation m, (ft)
Highest observed water level (08/08/83)	+2.38 (7.81)
Mean Higher High Water (MHHW)	+1.64 (5.39)
Mean High Water (MHW)	+1.41 (4.63)
Mean Sea Level (MSL)	+0.85 (2.78)
Mean Tide Level (MTL)	+0.85 (2.78)
National Geodetic Vertical Datum – 1929 (NGVD)	+0.78 (2.56)
Mean Low Water (MLW)	+0.28 (0.92)
Mean Lower Low Water (MLLW)	0.00
Lowest observed water level (12/17/33)	-0.79 (-2.60)
Elevations are referred to MLLW.	

2.2.2 Storm Surge

25. Storm surge is the super-elevation of the tidal level at the coast due to wind stresses and atmospheric pressure fluctuations acting upon the sea surface. Wind and atmospheric fluctuations associated with strong storms in southern California typically produce 0.3-0.6 m (1-2 ft) storm surges (CCSTWS). Due to a narrow continental shelf and the absence of tropical storms and/or hurricanes, storm surge heights on the California coast are small compared to those on the east and Gulf coasts where extreme surge heights of 1-3 m (3-10 ft) are more typical and a peak 8 m (25 ft) was documented during Hurricane Camille in 1969. West coast storm surges typically have time scales of 1-3 days, with longer surge episodes possible due to bunching of successive storms.

2.2.3 Relative Sea Level Change

26. The U.S. Army Corps of Engineers considers potential relative sea level change in every feasibility study undertaken within the coastal zone. Corps of Engineers policy guidelines for sea level rise is defined in Engineer Circular 1105-2-186 (Dept. of Army 1989) and a Department of Army letter (Dept. of Army 1986). Historic regional sea level trends based on yearly mean sea level records are published by the National Ocean Service (NOS) (National Ocean Service, 2001). Monthly mean sea level variations are analyzed for 117 stations of the NOS National Water Level Observation Network having between 25

and 146 years of data. Monthly MSL data are used to obtain the average seasonal cycle, the residual time series, and the autoregressive coefficient of the residual with accurate estimates of standard errors. Historic trends at San Diego, California indicate a positive sea level rise of +2.45 mm/yr based on water level measurements during the period 1950-1999. If past trends are projected into the future at San Diego, a sea level rise of 0.12 m (0.4 ft) would be expected over the next 50 years.

27. Relative sea level rise has project impacts from two primary considerations: 1) long-term beach erosion, and 2) increased wave runup and overtopping. The effects of sea level rise are addressed separately in Section 2.5.9.

2.3 Topography / Marine Geophysical

2.3.1 Topography

28. Terrestrial topographic data were obtained from March 2002 aerial LIDAR surveys conducted as a part of this study. LIDAR (LIght Detection And Ranging) is a state-of-the-art survey system that allows high-speed collection of topographic data. The system employs a helicopter-mounted range-finding laser that is coupled with a highly accurate GPS positioning system to collect precise GPS measurements, platform attitude, laser ranges, and imagery data. The LIDAR system collects terrain information at a rate of 10,000 points per second. A full description of the survey is described in Chance (John Chance, 2002).

29. Topographic information was collected at a horizontal point spacing on order of 0.1 meter that allowed detailed information to be collected of the beach, revetment, railroad, and near structure ground elevations. High-resolution topographic data allowed the development of high-resolution digital terrain models of all project features. Detailed mapping in the damage/flood areas provided existing beach contours, beach widths, berm elevations, foreshore slopes, and structure pad horizontal locations. Detailed mapping of the revetments and railroad provided accurate crest elevations.

30. The topographic data was adjusted for datum consistency with the marine survey data. The LIDAR topographic data was collected in the North American Vertical Datum of 1988 (NAVD88). It is usual and customary that coastal engineering projects are relative to an ocean-related datum, typically Mean Sea Level or Mean Lower Low Water. Survey control information indicates that NAVD88 = +0.14 m (+0.45 ft) MLLW at La Jolla, CA and NAVD88 = +0.12 m (+0.38 ft) MLLW at Newport Bay Harbor Entrance. Therefore, it was assumed NAVD88 = +0.13 m (+0.41 ft) at San Clemente and all topographic (LIDAR) measurements were adjusted down by –0.12 m (-0.41 ft).

2.3.2 Marine Geophysical

31. The marine geophysical data used in this analysis were obtained from June 2002 nearshore surveys conducted as a part of this study. A multi-sensor survey was conducted to collect bathymetry, side scan sonar, and sub-bottom geophysical information; a full description of the survey is described in Fugro (Fugro, 2002). The multibeam bathymetric survey provides point elevation data of the seabed in order to develop an accurate digital terrain model. The subbottom profile survey was conducted to delineate stratigraphic conditions, the presence or absence of sandy materials, the boundaries separating the consolidated and unconsolidated materials, define the depth to shallow bedrock if such exists, identify faults, paleochannels, and other anomalous geologic conditions within the survey area. The side scan sonar survey was used to acquire sonar imagery of the seabed in order to distinguish and map features/types and correlate surficial sediment types with the geophysical subbottom system. Sediment grab samples were collected for correlation with the survey data. The survey covered a corridor of approximately 14 km alongshore from approximately San Juan Creek near Dana Point to San Mateo Creek by 2.25 km cross-shore from the surf zone to 2500 meters offshore in approximately 23 meters water depth. The survey utilized closely spaced survey lines and resulted in the collection of approximately 534 km of geophysical data.

32. A full geological interpretation of the results can be found in the Geotechnical Appendix; a summary of some of the results is discussed below.

33. Bathymetry. The water depths in the survey area range from 3 meters near the beach to 23 meters offshore. The seafloor slope direction is southwest – normal to the beach. The seafloor gradient averages 0.9% (slope 110H:1V) but varies locally. The inshore gradient between 3-6 meters is 5% off San Clemente State Beach. This gradient decreases farther to the northwest.

34. Several bedrock spurs extend out from shore; the largest one is the seaward extension of the San Mateo Point. These spurs may rise several meters above the intervening swales. The San Mateo Rocks northwest of San Mateo Point are isolated and may be remnant spurs. Bedrock outcrops the seafloor in places between the shore and about the 15-meter isobath. Where outcrops occur, the seafloor is uneven from the resistant bedrock mounds. Some of the larger outcrops rise as much as 6 meters above the surrounding seabed. The gradients along some of these outcrop slopes can be as high as 33% (18^o). A smooth seafloor with an even slope forms the topography seaward of the outcrops. This smooth texture is a result of unconsolidated recent sediment deposition.

35. Surficial Features and Sediment Thickness. The side scan sonar data clearly show areas where bedrock is exposed. In several locations, survey data could not be acquired as the kelp was too thick to navigate through. It is well established that bedrock is necessary for kelp growth. The bedrock exposures are mapped as either areas where exposures comprise greater than 50% of the seabed and zones where scattered rock covers 10-50% of the area.

36. Unconsolidated surficial sediment predominates in the scattered rock zones. The subbottom profile data reveal an immeasurably thin surficial veneer overlies the bedrock. This surficial sediment cover likely changes seasonally as beach sands migrate cross-shore. It is expected that within the shallower water depths where sediment caps the bedrock, the sediment thickness is probably less than 0.3 meter (1 foot).

2.4 Wave Climatology

37. Wave climatology information is available for the offshore area of San Clemente in the form of direct measurements as part of the Coastal Data Information Program (CDIP).

38. The CDIP shallow-water station most applicable to San Clemente is the San Clemente S_{xy} slope array (Station ID 052) located approximately 300 m offshore of the San Clemente Pier in 10.2 meters of water. The S_{xy} slope array is a directional wave height recorder with a 178-month record during the period 1983-1998 at nominally four observations per day. The height and direction data records are intermittent reporting approximately 141 months of the 178 available; one long gap occurring during the period Jul88-Jul91 accounting for the majority of missing records. These data records are used to determine the daily and annual wave climate, the extreme wave analysis, and data input into wave transformation and shoreline change numerical models.

39. Determination of the annual wave climate entailed assembling the entire data records. These records are sorted based on height, period, and direction. The resultant univariate and multivariate distributions are discussed and shown hereinafter.

2.4.1 Wave Heights

40. The histogram for significant wave height is shown in Figure 2. Several observations can be made regarding the wave climate. Figure 2 illustrates that the most commonly occurring significant wave height is in the range 0.80-1.00 m. Figure 2 also illustrates the complete absence of very large significant wave heights (> 4.0 m). The maximum recorded significant wave height is 3.63 m. This may be due to several conditions including the fact that the S_{xy} slope array measures the wave climate in 10 m of water, whereas waves observed closer to the breaking depth will be larger due to shoaling. Also, the wave data are spectral-energy based (H_{mo}) rather than statistically based ($H_{1/3}$), it is well established that individual wave heights within the wave train represented by H_s (H_{mo}) are substantially larger than the H_s value. The

wave climate is measure in the vicinity of the pier and is assumed representative of the entire San Clemente project area.

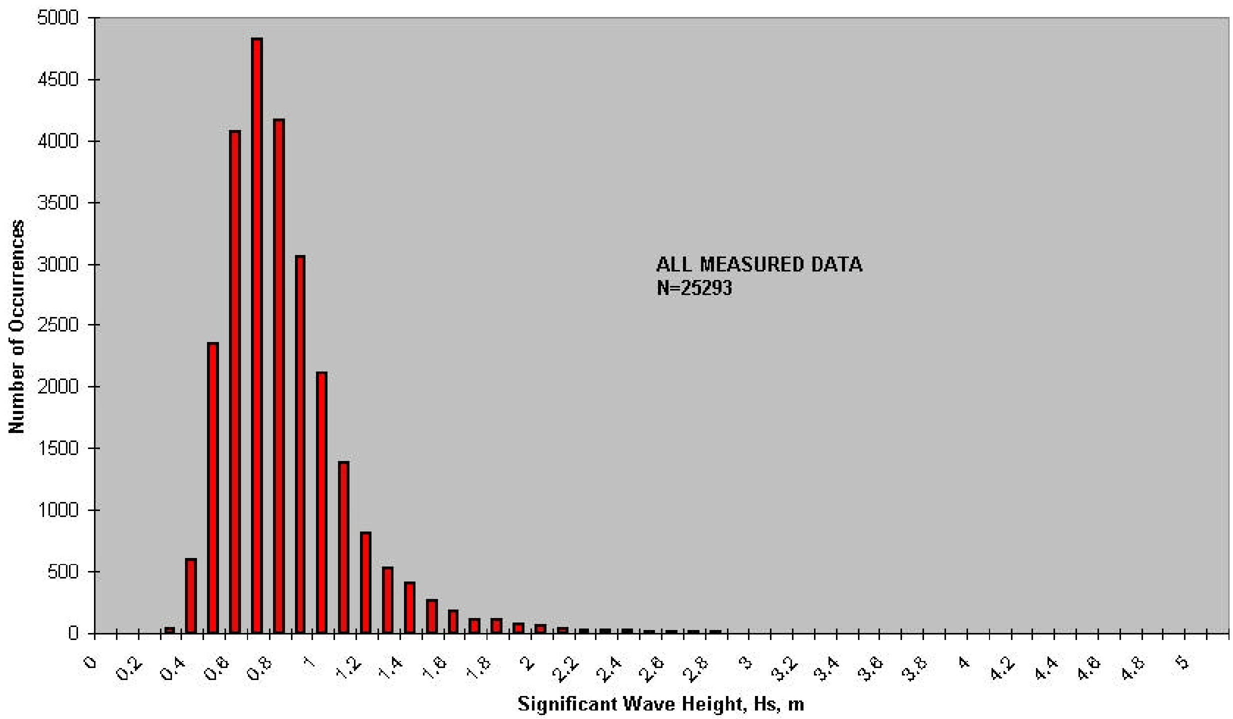

Figure 2 Significant Wave Height Histogram, 1983-1998

41. The winter wave climatology can be developed from the measured wave climate previously discussed. It is recognized that the severest wave climate occurs during the winter season. It was desired to develop the wave climatology based on the winter wave population. The wave data was categorized based on the December-March winter meteorological season. The histograms for the winter significant wave height are shown in Figure 3.

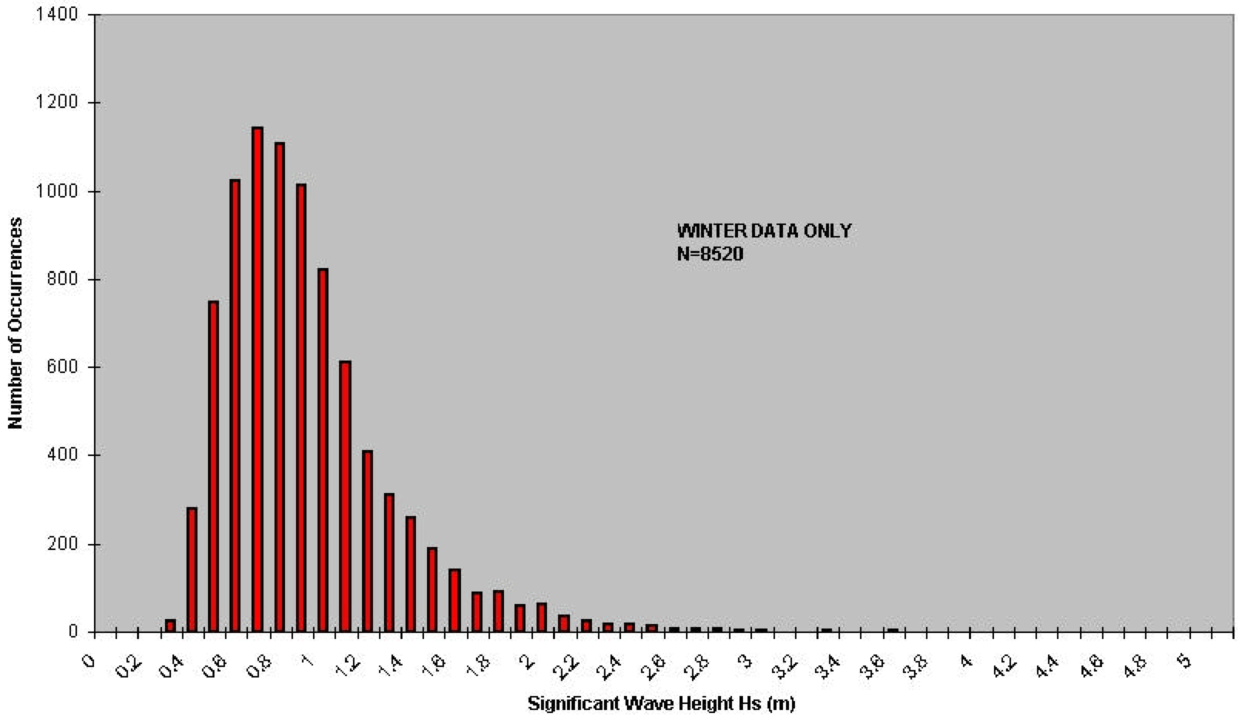

Figure 3 Significant Wave Height Histogram, Winter Data, 1983-1998

42. Based on the measured data, the annual maximum wave heights for each year were determined. The annual maximum wave heights are shown in Table 2.

Table 2 Annual Maximum Wave Heights, 1983-1998

Year	Mo/Dy	Hs, m (ft)	Tp (sec)	Dir (deg)
1983	Dec 10	3.1, (10.2)	14.2	248
1984	Apr 01	1.85, (6.1)	6.1	273
1985	Nov 29	2.18, (7.2)	6.1	227
1986	Feb 16	3.56, (11.7)	16	234
1987	Mar 16	2.24, (7.4)	8.5	254
1988	Jan 18	3.63, (11.9)	14.2	218
1991	Nov 15	2.06, (6.8)	18.3	212
1992	Jan 30	2.32, (7.6)	14.2	250
1993	Feb 18	2.66, (8.7)	8	197
1994	Feb 07	2.0, (6.6)	7.3	191
1995	Jan 05	3.22, (10.6)	9.1	205
1996	Oct 26	2.24, (7.4)	8.8	259
1997	Dec 06	2.31, (7.6)	6.4	207
1998	Jan 30	2.99, (9.8)	18.3	238

2.4.2 Wave Periods

43. Figure 4 illustrates that the dominant wave periods are in the range 12-14 seconds, with a smaller secondary peak at 6-8 seconds. The two peaks in the distribution demonstrate the dual sea/swell nature of the wave climate. Shorter period waves are typically associated with sea conditions; longer period waves are typically associated with swell conditions.

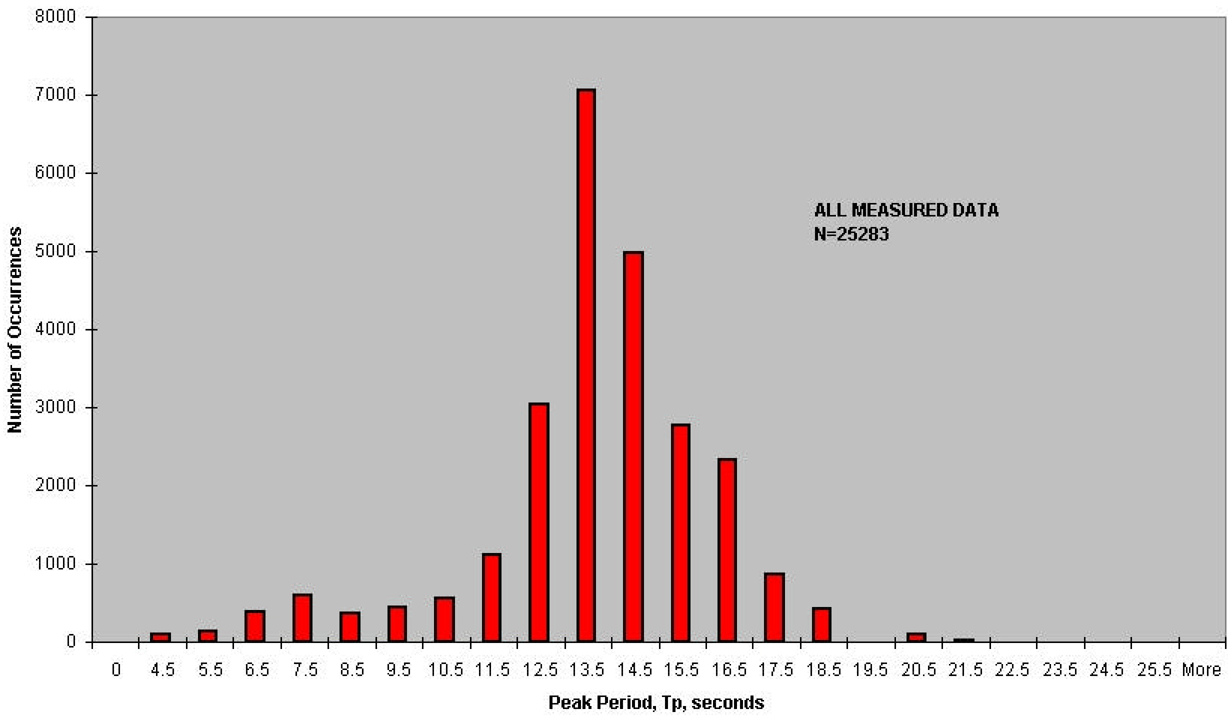

Figure 4 Spectral Peak Period Histogram, 1983-1996

2.4.3 Wave Directions

44. Directional separation of the data results in Figure 5. This figure illustrates that approximately 91% of the waves approach from the relatively narrow 20-degree band between 230º-240º, and all other approach directions are minor or negligible. Shoreline normal in the San Clemente area is 235º. There is a very small fraction of waves (0.7%) approaching from 160º-220º, directions considered to be of tropical depression or southern hemisphere origins. Significantly, the predominant westerly wave direction envelops both local seas and extratropical swell.

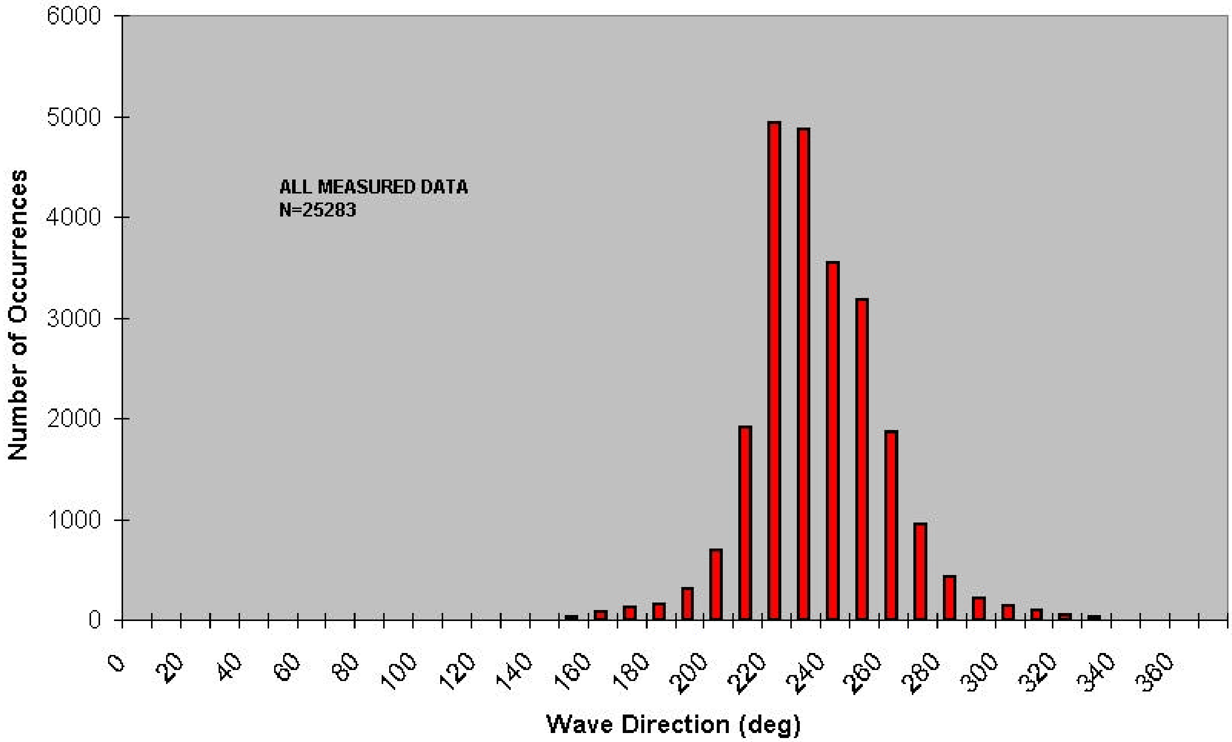

Figure 5 Wave Direction Histogram, 1983-1996

2.5 Littoral Processes and Sediment Transport

45. This section describes the sediment budget analysis, the shoreline change analysis, other related input data analysis, and the effects of sea level change for the San Clemente shoreline study area.

2.5.1 Oceanside Littoral Cell

46. The Oceanside Littoral Cell extends for 86 km (53.5 mi) from Dana Point to Point La Jolla. The shoreline displays a wide variety of coastal features including cliffs, headlands, beaches composed of sand and/or cobbles, rivers, creeks, tidal lagoons and marshes, submarine canyons, man-made shore and bluff protection devices of many kinds, and major harbor structures. The cell is divided into three sub-cells based on natural physiographic units: (1) Dana Point to San Mateo Point, (2) San Mateo Point to Carlsbad Submarine Canyon, and (3) Carlsbad Submarine Canyon to Point La Jolla. The City of San Clemente is located in the north sub-cell.

47. The littoral cell includes two small craft harbors, Dana Point Harbor and Oceanside Harbor. Dana Point Harbor is located on the northern end of the littoral cell, while Oceanside Harbor is in the center of the cell.

2.5.2 Sediment Sources

48. Numerous rivers and small streams discharge sediment into the Oceanside Littoral Cell, as shown in Figure 6. San Juan Creek and San Mateo Creek are considered major river systems for the influx of

sediment into the north sub-cell. Extracts from a listing compiled from the results of various studies (CCSTWS) is presented in Table 3 and provides a range of estimates of the sediment loads carried by fluvial systems of this littoral sub-cell. This list is supplemented with a recent study for the San Juan Creek watershed.

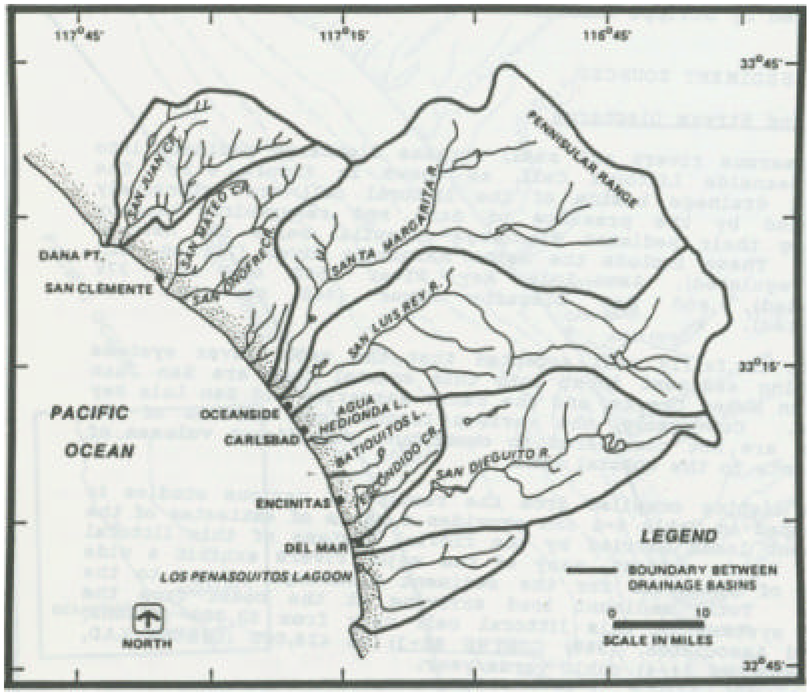

Figure 6 Major Rivers and Drainage Basins, Oceanside Littoral Cell

Table 3 Sediment Discharge from Rivers and Streams

	River / Stream Discharge Rate m^3/yr (yd^3/yr)		
	San Juan	San Clemente	San Mateo
Drainage Area (hect / mi2)	45455 (175.5)	5154 (19.9)	34188 (132)
Moffatt&Nichol 1977	12920 (17000)	10898 (14340)	1702 (2240)
CCSTWS 84-4 (1984)			24320 (32000)
Simons/Li 1985	6080 (8000)		12160 (16000)
CCSTWS 88-3 (Simons/Li 1988)	15534 (20440)	783 (1030)	3713 (4885)
CCSTWS 90-2 (Moffatt&Nichol 1990)	27360 (36000)		6384 (8400)
COE-LAD 1999	39749 (52071)		

2.5.3 Sediment Budget

49. A sediment budget for without-project conditions has been developed based on the CCSTWS. Development of the sediment budget involves defining the sediment sources, sinks, and losses; transport modes; erosion and accretion rates; and balancing the resultant budget. Some additional information was obtained during this study to enhance the previously developed sediment estimates. Compilation of the sediment budget specific to San Clemente is described hereinafter and is further described in the CCSTWS.

50. The analysis of the budget of sediment for this cell has been carried out for three time periods: (1) the period from 1900 – 1938, (2) a mild, uniform weather period from 1960 – 1978, and (3) a period of more variable wave climate covered by the CCSTWS studies from 1983 – 1990. The 1900 – 1938 "natural" budget permits an uncluttered look at the cell as it predates construction of dams and Oceanside Harbor, although it necessarily draws on some findings from later studies. The mild, uniform period from 1960-1978 was selected to evaluate the effects of Oceanside Harbor at a time when the wave climate was consistent from year to year and less variable than the present wave climate. The last period of more variable wave climate 1983 – 1990 emphasizes the change in wave climate from one that gave a consistent, strong southerly littoral transport to one that yields a more variable transport with a net northerly component in some years. The resultant sediment budget for the three time periods is shown in Table 4.

Table 4 Sediment Budget for Oceanside Littoral Cell (North – San Clemente) in 1000 m^3 / yr

	1900-1938		1960-1978		1983-1990	
	Input	Output	Input	Output	Input	Output
Q_l	0	130	0	130	0	35
Q_n	0	15	0	15	0	5
$Q_{b,o}$	130	45	90	45	45	0
Q_a	0	0	90	0	0	0
$Q_{r,s}$	65	0	45	0	0	0
Total	+195	-190	+225	-190	+45	-40
Net $(\partial V'/\partial t)$	+5		+35		+5	
$\partial X/\partial t$ (m/yr)	+0.03		+0.18		+0.03	

Notes:

Q = total sand transport rate into or out of cell, m^3/yr
a = artificial nourishment, bypassing, dredging, etc
b = blufflands erosion; includes seacliffs, gullies, coastal terrace, slumps, etc as distinct from rivers
l = longshore transport of sand in and near the surfzone
n = nearshore transport along the coast, outside the surfzone
o = onshore/offshore transport at the base of the shorerise

r = river yield to the coast
s = lost to submarine canyons

$\partial V' / \partial t$ = sand volume change rate, m^3/yr

$\partial X / \partial t$ = shoreline change rate, m/yr

51. The resultant sediment budget indicates the shoreline is essentially in balance between erosion and accretion. The budget is considered to be in balance when the shoreline change rate $\partial X / \partial t$, computed from the volume flux is less than 0.03 m/yr (0.1 ft/yr). The shoreline indicates a balance in the "natural" time period and the most recent variable wave climate time period. The net volume flux indicates the budget is very slightly accretional during the uniform wave climate period.

2.5.4 Long Term Shoreline Change

Historical Shoreline Change

52. Shoreline changes within the Oceanside Littoral Cell were investigated by the CCSTWS using historical maps and nautical charts, aerial photos, and the results of ground and bathymetric survey efforts. The results of these extensive efforts are shown in Table 5.

53. The table indicates the alongshore variation of shoreline change within the immediate vicinity of San Clemente; the San Clemente study area is between SC 1623 (State Beach) and SC 1720 (Shorecliffs). There are contradictory trends observed in the data. The data sets are out of phase with adjacent locations; a transect is erosional and/or accretional while the adjacent transect is accretional and/or erosional. The mean values during the 1940-1960 and 1960-1980 periods are similar in magnitude; the mean values during the 1980-1989 period are remarkably higher. Detailed inspection of the data indicates a shoreline that is either erosional, balanced, or accretional. During the period 1940-1960, the shoreline indicated essentially zero change with a +0.76 m/yr change in vicinity of SC 1680. During the period 1960-1980, the shoreline vacillated alongshore between positive and negative. The shoreline change was approximately equal between positive and negative ranging from -0.21 m/yr and +0.18 m/yr. During the period 1980-1989, the shoreline was predominantly positive with accretion rates ranging from +0.43 m/yr to +2.16 m/yr; an erosion value of –0.61 m/yr was recorded at SC 1660.

Table 5 Long Term Shoreline Change Rates in San Clemente Area

	MHHW Shoreline Change Rate, m/yr (ft/yr)			Max Seasonal MHHW Movement, m (ft)	
Location	1940-1960	1960-1980	1980-1989	Summer	Winter
SC 1623	-0.06 (-0.20)	-0.21 (-0.70)	2.16 (7.10)	7.7 (25.4)	-7.9 (-26)
SC 1660	0.00 (0.00)	0.18 (0.60)	-0.61 (-2.00)	5.2 (17)	-10.4 (-34)
SC 1680	0.76 (2.50)	-0.12 (-0.40)	0.43 (1.40)	13.9 (45.5)	-17.5 (-57.4)
SC 1720	0.00 (0.00)	0.00 (0.00)	1.46 (4.80)	9.2 (30)	-8.2 (-27)
DB 1805	-0.58 (-1.90)	2.47 (8.10)	-3.75 (-12.30)	7.6 (25)	-13.9 (-45.6)
DB 1850	-0.18 (-0.60)	2.84 (9.30)		0.8 (2.7)	-21.4 (-70.2)
DB 1895	0.76 (2.50)	-0.12 (-0.40)	-0.15 (-0.50)	7.5 (24.6)	-9.6 (-31.4)
DB 1900	0.00 (0.00)	-0.58 (-1.90)	-3.05 (-10.00)	18.2 (59.8)	-27.9 (-91.4)

Current Beach Width Monitoring

54. The City of San Clemente initiated a beach monitoring program as part of the non-Federal in-kind contributions (Coastal Frontiers, 2002). The general objective of the monitoring program is to document changes in the condition of the shore between Dana Point Harbor and San Mateo Point thereby providing a basis for evaluating the impacts of natural events and anthropogenic operations. The program includes semi-annual full cross-shore profile surveys at 11 representative sites and bi-monthly beach width measurements at 9 of the 11 profile sites. The full cross-shore profiles were obtained by contract whereas City of San Clemente lifeguards obtained the bi-monthly beach width measurements.

55. A description of the transect locations is given in Table 6. The 11 profile locations include 6 historical locations originally established by the CCSTWS, and 5 locations established specifically for the beach monitoring program.

Table 6 San Clemente Area Beach Profile Transects

Site #	Transect Designation	Location	Origin
1	DB-1850	N. Doheny State Beach	CCSTWS
2	DB-1805	N. Doheny State Beach	CCSTWS
3	SC-1720	Shorecliffs	CCSTWS
4	SC-1705	Capistrano Trailer Court	Est. Oct. 2001
5	SC-1700	North Beach	Est. Oct. 2001
6	SC-1695	Dije Court	Est. Oct. 2001
7	SC-1680	Linda Lane	CCSTWS
8	SC-1660	T-Street	CCSTWS
9	SC-1645	Lost Winds	Est. Oct. 2001
10	SC1623	San Clemente State Beach	CCSTWS
11	SC-1605	Cottons Point	Est. Oct. 2001

Recent Shoreline Change Rate

56. The shoreline change rate can be determined from the aggregate measured data collected in support of the CCSTWS and the City of San Clemente beach width monitoring program. The data is a compilation of measurements obtained in the 1980's thru the present.

57. It is noted that this beach width data set is expressed relative to the MSL contour as opposed to the berm definition adopted for this study. The beach widths are the distance between a fixed point on the backshore and the approximate location of the MSL contour which is a commonly accepted definition. This MSL beach width incorporates a portion of the "wet" beach e.g. the foreshore between the MSL contour and the berm whereas the berm beach width definition incorporates only the "dry" portion of the beach. Thus the MSL beach widths will be inherently greater than the berm beach widths. The MSL indicates a positive beach width where the beach has been previously defined in many reaches as having zero width. Based on a typical beach slope of 8H:1V and berm elevation of +6.2 m and a MSL contour elevation of +1.64 m, the estimated horizontal beach width attributable to this contour elevation difference is approximately 35 m (114 ft). Thus it can be shown that the dry beach width are zero at Linda Lane and near zero at the other locations. Thus the data sets are consistent with the conclusions developed in this study. It is assumed that the trend for the berm widths will coincide with the trend for the MSL data.

58. The measured data is shown for the four locations that are historical to the CCSTWS. The linear regression for each data set is shown. The slope of the lines represents the mean shoreline trend for each respective data set. The long-term shoreline change rate data sets are shown in Table 7. The shoreline change data sets are considered together to obtain representative values for the entire study

area. The mean shoreline change rate is –0.20 m/yr (-0.7 ft/yr), the maximum erosion rate is –0.61 m/yr (-2.0 ft/yr) and the maximum accretion rate is +0.38 m/yr (+1.24 ft/yr).

59. There are contradictory trends observed in the data. Shorecliffs is the data set that is out of phase with the other three. The three data sets around the pier are consistent in trend and phase. The data sets indicate consistent erosion and accretion trends at the same time. The mean values are similar in magnitude. The data set at Shorecliffs is nearly opposite in behavior. The beach is erosional and/or accretional when the others are accretional and/or erosional. The data in the 1980's is consistent among data sets. After the long absence of data in the 1990's, the 2002 data indicates a loss of beach width.

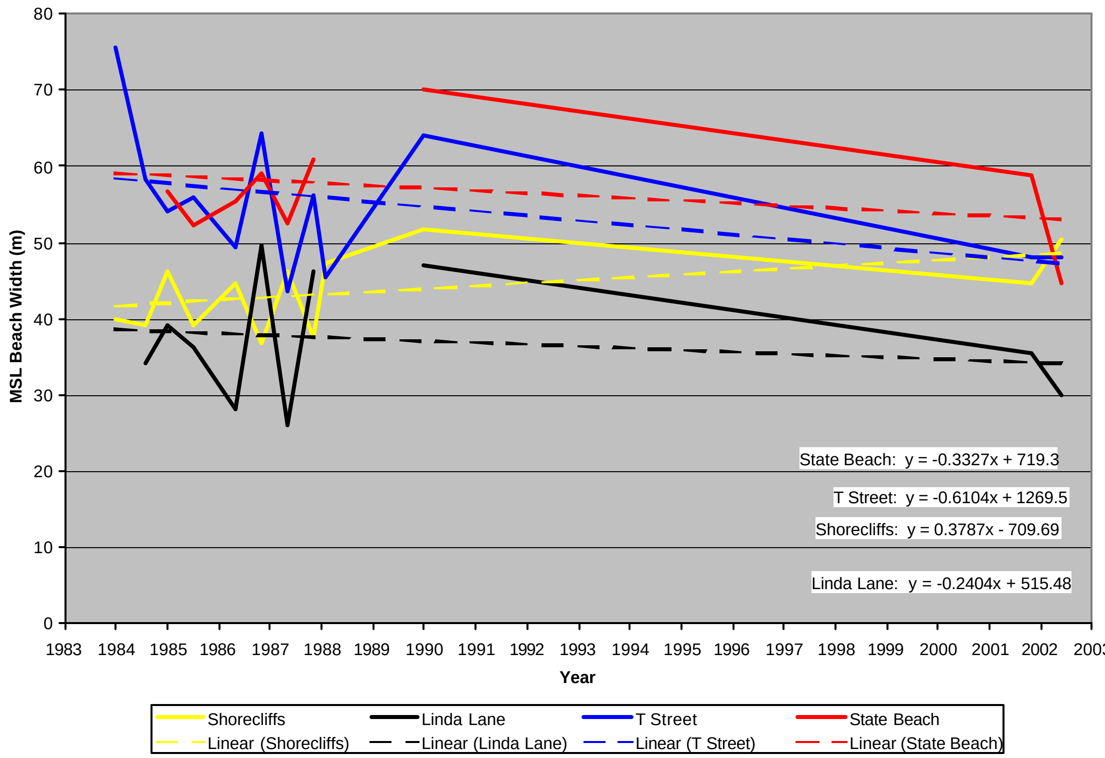

Figure 7 Recent Shoreline Change in San Clemente Area

Table 7 Summary of Recent Long Term Shoreline Change Rates

Location	Erosion Rate, m/yr (ft/yr)
SC 1720, Shorecliffs	+0.38 (+1.24)
SC 1680, Linda Lane	-0.24 (-0.79)
SC 1660, T – Street	-0.61 (-2.00)
SC 1623, State Beach	-0.33 (-1.09)

2.5.5 Cross-shore Profiles

60. Cross-shore profiles are compiled from the LIDAR topographic data and bathymetric measured data for Reaches 1-8. Profiles from Reach 6 (vicinity pier area) and Reach 7 (Mariposa Point) are representative of the beach and the armored shoreline respectively that are characteristic in the project area and are shown in Figure 8 and Figure 9. Only the portion of the profile from the bluff to the waterline is shown to better illustrate the detail of the foreshore and backshore regions. The profile centerline is established at the seaward rail of the SCRRA railroad.

61. The pier area beach profile indicates a typical berm elevation of +5.2 m (+17 ft), a typical foreshore slope of 8H:1V – 10H:1V, an offshore slope of 110H:1V, and a railroad elevation of approximately +6.4 m (+21 ft).

62. The Mariposa Point area profile indicates a mean revetment elevation of +6.9 m (+23 ft), typical revetment slope of 1H:1V, toe elevation at approximately 0.0 m, an offshore slope of 110H:1V, and a railroad elevation of approximately +6.4 m (+21 ft).

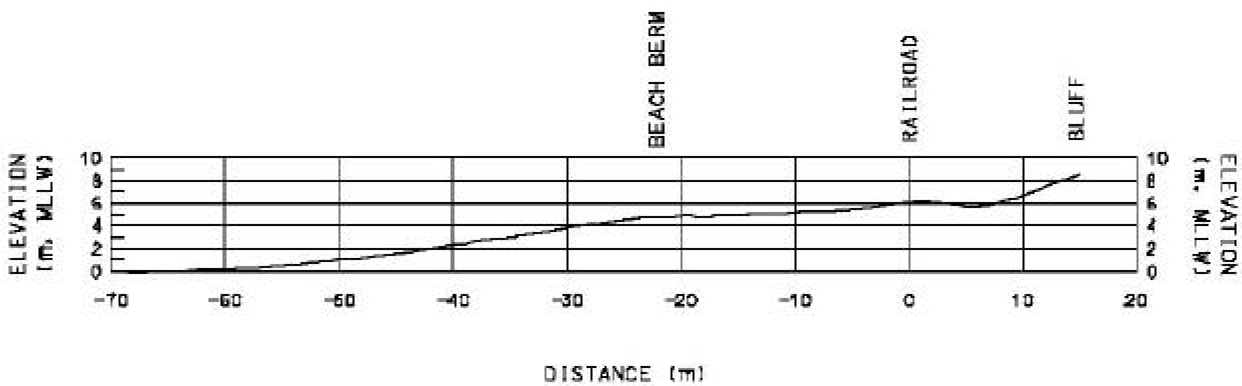

Figure 8 Typical Cross-shore Section for Reach 2, Reach 4, Reach 6, Reach 8 (No Revetment)

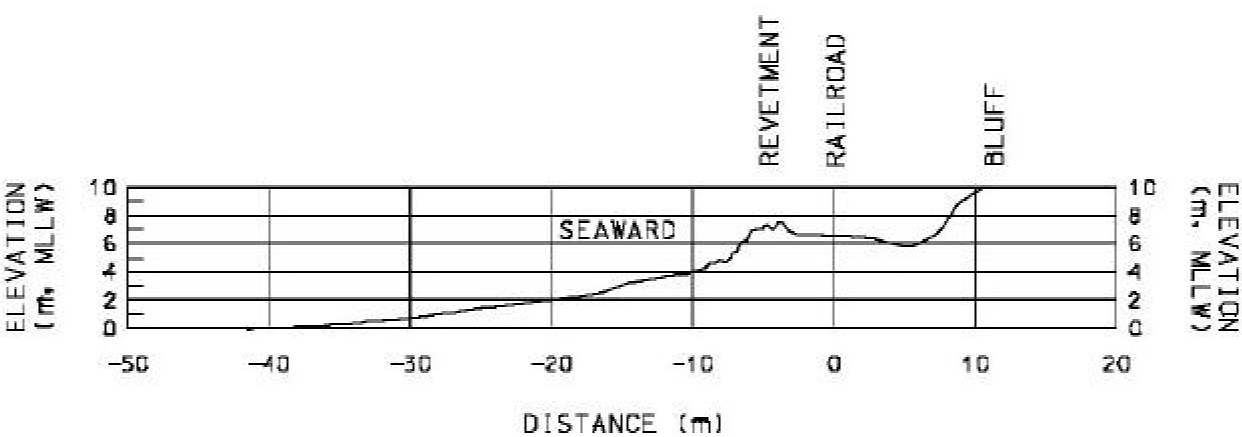

Figure 9 Typical Cross-shore Section for Reach 1, Reach 3, Reach 5, Reach 7 (Revetment)

2.5.6 Foreshore Slopes

63. Foreshore slope data was obtained from direct measurements. The City of San Clemente lifeguards obtained direct measurements of the foreshore slope as part of the aforementioned beach width monitoring program. Approximately 21 measurements were obtained 2-3 times each month for a 12 month duration during the period Nov 2001 – Nov 2002 at nine selected locations throughout the study area. The selected locations include all identified in Table 6 except DB 1850 and DB 1805. The slope was measured in degrees from horizontal and converted to the slope cotangent. Assuming the year time period is representative of the project period, this data set represents an annual variation of foreshore slope values. A portion of the foreshore slope data is shown in Table 8. The histogram of all foreshore measurements is shown in Figure 10.

Table 8 Beach Monitoring Measurements at T-Street

Date	Width	Slope (deg)	Slope (cotan)
Nov 26, 2001	152	8	7
Dec 13, 2001	152	3	19
Jan 07, 2002	132	4	14
Jan 22, 2002	158	3	19
Feb 07, 2002	151	5	11
Feb 20, 2002	176	5	11
Mar 06, 2002	193	5	11
Mar 22, 2002	155	5	11
Mar 31, 2002	115	5	11
Apr 15, 2002	136	5	11
Apr 30, 2002	150	6	10
May 14, 2002	149	7	8
Jun 07, 2002	143	6	10
Jun 28, 2002	200	7	8
Jul 12, 2002	203	11	5
Aug 01, 2002	139	7	8
Aug 16, 2002	179	7	8
Aug 29, 2002	135	5	11
Sep 06, 2002	172	9	6
Sep 26, 2002	125	9	6
Nov 16, 2002	142	7	8

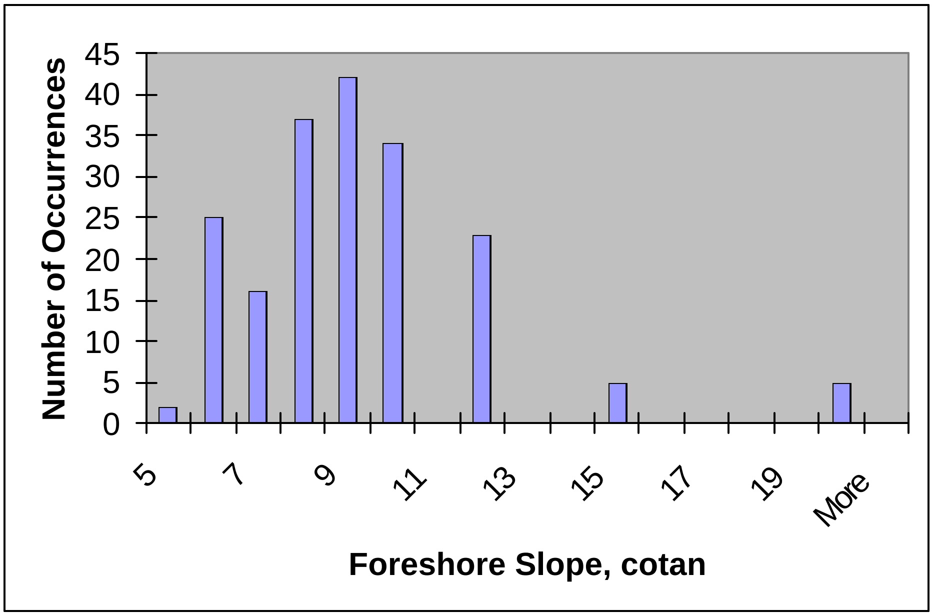

Figure 10 Foreshore Slope Histogram

2.5.7 Storm Induced Beach Change

64. Short-term shoreline erosion data have been collected within the Los Angeles District as part of the Orange County Beach Erosion Control Project (Surfside-Sunset). This data set represents a collection of linear beach widths collected at 26 locations over a period of 33 years. This data set is used in the present analysis to estimate shoreline response under storm conditions.

65. The data set is collected at 26 locations representing various beach and shoreline conditions. Locations are selected which are deemed to be largely free of the influence of coastal structures and their attendant effects on shoreline response. Ten known significant storms were selected for analysis. This results in a subset of measured shoreline response data that is directly correlated to storm input. The aforementioned study area is morphologically very different from the San Clemente study area. The northern Orange County area is primarily wide sandy beaches and a full sand profile. This is compared to the San Clemente study area that has been shown to be primarily a hard bottom area within a thin lens of sand along the shoreline. Thus the San Clemente area has inherently less beach width to exchange cross shore due to storm response. Therefore, the raw data collected from northern Orange County was modified to more realistically reflect the expected San Clemente shoreline response. The results of this analysis are shown in Figure 11.

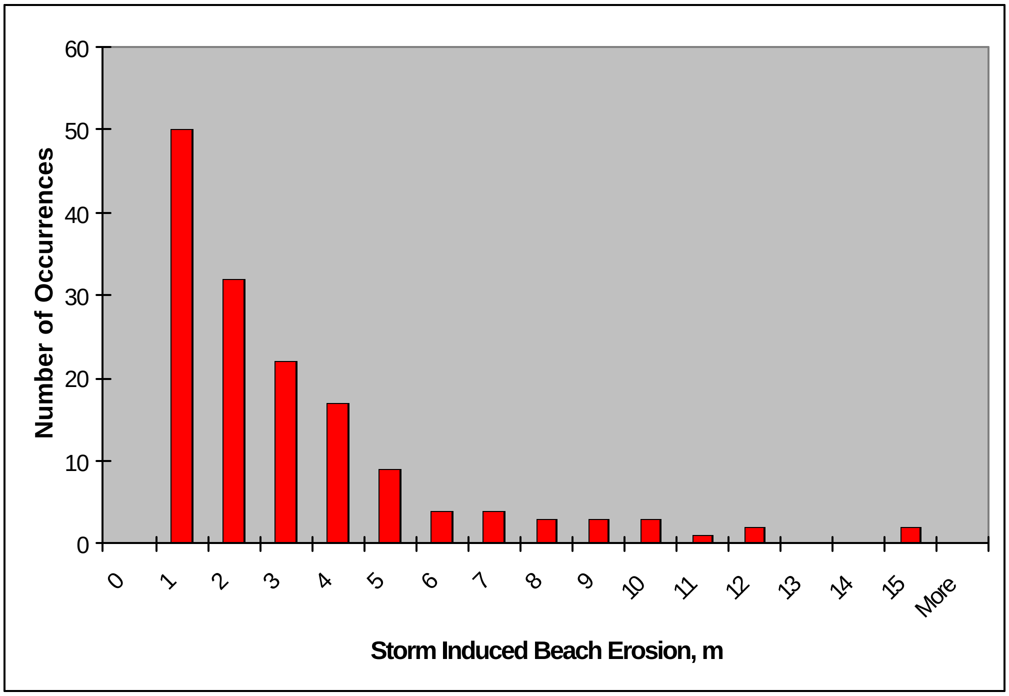

Figure 11 Storm Induced Beach Change Histogram

2.5.8 Sediment Profile Thickness

66. Data collected for the Sand Thickness Survey Report (USACE-LAD 1988) allows estimation of the available sediment supply and consequently any potential limits to erosion. This study conducted jet probing along various profiles to determine the available sediment thickness. Three profiles in the San Clemente area were jet probed including SC-1623 (San Clemente State Beach), SC-1660 (T-Street) and SC-1720 (Capistrano Shores). The survey results indicate the sediment thickness cross-shore along the profile. The results are summarized in Table 9.

67. On profile SC-1623, the sediment thickness ranges from 0.1-3.2 meters (0.3-10.5 ft). The average profile is consistently thick 2.4-3.2 m (7.7-10.5 ft) across the beach locations and is near zero 0.1-0.6 m (0.4-1.8 ft) across the seaward portion of the profile.

68. On profile SC-1660, the sediment thickness ranges from 0.0-4.5 meters (0.0-14.8 ft). The average profile is consistently thick 2.1-4.5 m (6.9-14.8 ft) across the beach locations and is near zero 0.1-0.7 m (0.4-2.2 ft) across the inner seaward portion of the profile. The data indicates a thickness of 2.7 m (8.8 ft) at the furthest offshore location.

69. On profile SC-1720, the sediment thickness ranges from 0.0-4.6 meters (0.0-15.0 ft). The average profile is consistently thick 1.0-4.5 m (3.3-14.9 ft) across the beach locations and is near zero 0.0-0.2 m (0.0-7 ft) across the seaward portion of the profile.

70. The measurement results identified cobbles, boulders, and other hard substrate at various depths along the profile. The observations include "some pebbles scattered on beach surface and some boulders visible at backshore" and "offshore sand-stone outcrops with local bottom relief of 1 ft". This information is consistent with 2002 geologic information collected during geophysical studies conducted as part of this study, and reported in the Geotechnical Appendix.

Table 9 Summary of Sand Thickness

	Probe Location			Sand Thickness		
Range Line	No.	Range	Elevation (MLLW)	1	2	Average
SC-1623	1	21.1 m (69.3 ft)	4.1 m (13.5 ft)	3.0 m (9.9 ft)	3.1 m (10.2ft)	3.1 m (10.1ft)
SC-1623	2	34.6 m (113.5 ft)	3.4 m (11.1 ft)	3.2 m (10.5ft)	3.2 m (10.4ft)	3.2 m (10.5ft)
SC-1623	3	51.7 m (169.6 ft)	2.1 m (6.9 ft)	2.4 m (7.7 ft)	2.3 m (7.6 ft)	2.4 m (7.7 ft)
SC-1623	4	194.2 m (636.8 ft)	-3.5 m (-11.4 ft)	0.1 m (0.3 ft)	0.1 m (0.4 ft)	0.1 m (0.4 ft)
SC-1623	5	266.1 m (872.8 ft)	-6.0 m (-19.6 ft)	0.2 m (0.8 ft)	0.3 m (0.9 ft)	0.3 m (0.9 ft)
SC-1623	6	504.2 m (1653.8 ft)	-9.3 m (-30.5 ft)	0.6 m (1.8 ft)	0.6 m (1.8 ft)	0.6 m (1.8 ft)
SC-1660	1	11.5 m (37.6 ft)	5.0 m (16.3 ft)	4.5 m (14.8ft)	4.5 m (14.8ft)	4.5 m (14.8ft)
SC-1660	2	23.5 m (77.2 ft)	3.1 m (10.2 ft)	3.4 m (11.0ft)	3.5 m (11.4ft)	3.4 m (11.2ft)
SC-1660	3	42.9 m (140.6 ft)	1.5 m (4.8 ft)	2.2 m (7.2 ft)	2.0 m (6.6 ft)	2.1 m (6.9 ft)
SC-1660	4	232.8 m (763.5 ft)	-3.1 m (-10.1 ft)	0.0 m (0.0 ft)	0.2 m (0.8 ft)	0.1 m (0.4 ft)
SC-1660	5	462.4 m (1516.5 ft)	-6.5 m (-21.3 ft)	0.6 m (2.0 ft)	0.7 m (2.3 ft)	0.7 m (2.2 ft)
SC-1660	6	673.6 m (2209.5 ft)	-9.0 m (-29.6 ft)	2.6 m (8.5 ft)	2.7 m (9.0 ft)	2.7 m (8.8 ft)
SC-1720	1	10.9 m (35.9 ft)	4.7 m (15.4 ft)	4.6 m (15.0ft)	4.5 m (14.8ft)	4.5 m (14.9ft)
SC-1720	2	24.8 m (81.4 ft)	2.7 m (8.7 ft)	2.8 m (9.2 ft)	3.0 m (9.9 ft)	2.9 m (9.6 ft)
SC-1720	3	46.5 m (152.6 ft)	0.6 m (1.9 ft)	1.0 m (3.3 ft)	1.0 m (3.3 ft)	1.0 m (3.3 ft)
SC-1720	4	165.2 m (541.9 ft)	-2.5 m (-8.1 ft)	0.0 m (0.0 ft)	0.0 m (0.0 ft)	0.0 m (0.0 ft)
SC-1720	5	494.8 m (1622.9 ft)	-6.7 m (-21.9 ft)	0.2 m (0.5 ft)	0.2 m (0.5 ft)	0.2 m (0.5 ft)
SC-1720	6	879.5 m (2884.9 ft)	-8.6 m (-28.3 ft)	0.2 m (0.5 ft)	0.2 m (0.8 ft)	0.2 m (0.7 ft)

2.5.9 Sea Level Rise

71. Relative sea level rise is considered to have an indirect impact on the 50-year future without-project and future with-project conditions. Relative sea level rise can be significant for long-term beach erosion rates. As the relative water level rises, a landward migration of the shoreline can be expected assuming the profile shape does not change and the longshore sediment transport is in equilibrium (i.e., no erosion or deposition). However, this type of analysis is not a substitute for an analysis of historical shoreline and profile change when the necessary data are available. The aforementioned CCSTWS historic sediment transport analyses have identified the expected shoreline change rate based on measured data and oceanographic processes. These analyses were based on data time scales similar to the 50-year life of this project. These analyses are considered more accurate and representative than a generalized calculation for long-term erosion rates based solely on sea level rise.

3 DESIGN CONDITIONS AND METHODOLOGY

3.1 Damage Mechanisms

72. Coastal storm damages at San Clemente consist of: direct structure damages due to wave impact forces, rail traffic service delays due to wave overtopping, and loss of recreational beach due to long-term beach erosion. San Clemente has been subdivided into 10 sub-reaches as described in Section 2.1. The separate sub-reaches are treated independently in the without (and with) project conditions with respect to the damage mechanisms experienced by each and the functional requirements of the individual analysis procedures as they are applied. The primary distinction between sub-reaches 1-8 is the absence or presence of armor stone revetment improvements on the railway.

73. Structure damages are confined to revetments protecting the railway. Structure damages are not considered north of the Capistrano Shores development as the railway is landward of this development. Damages are not considered south of San Mateo Point.

3.1.1 Wave Impact Forces

74. Structure damages from wave impact forces begin when wave propagation (or wave runup) intersects the structure seaward boundary. The degree of impact loading is a function of the runup elevations and the structure spatial positions. Wave force damages are treated independently and equally for each sub-reach.

75. Structure damages due to wave runup were estimated based on the damage functions shown in Table 10. The damage relationship is based on runup elevation in lieu of wave height. It is noted that the majority of the railway and associated revetments are above the MHHW elevation. As such, the revetments are not directly exposed to wave heights (such as the case for an offshore structure), but are directly exposed to wave runup. Incorporating the use of runup elevation allows for consideration of still water level, which has a significant effect on the computed runup elevation.

76. The percent damage is based on the quantity of armor unit removal for a specific runup elevation (wave height). These damage functions are similar in concept and nature to the percent damage and no-damage criteria described in the Shore Protection Manual (SPM) (USACE-CERC, 1984). A distinction was made between the two primary types of railway protection: a) ballast only (no protection); and b) non-engineered which defines the protection provided by the large armor stone dumped in an uncontrolled (non-engineered) fashion. A 5% no-damage threshold was applied; damages greater than 5% were accrued. The damage function relativity between the ballast only and non-engineered railway was adjusted and calibrated so that predicted damages in the initial year of the life-cycle simulation approximately matched the existing damages experienced by the SCRRA.

Table 10 Railroad Damage Functions

Runup Elev, m (ft)	% Damage for Ballast RR	% Damage for Non-Engineered RR
0	0	0
3.05 (10)	0	0
3.66 (12)	1	0
4.27 (14)	5	0
4.88 (16)	10	1
5.49 (18)	15	5
6.10 (20)	20	10
7.62 (25)	25	15

3.1.2 Railway Traffic Service Delay

77. Railway traffic service delays result when excessive wave runup elevations and wave overtopping result in a flow rate of water that exceeds the elevation of SCRRA protective revetments. According to the SCRRA, railway traffic service delays have occurred during severe storms due to wave action. The SCRRA has documented two service disruption incidents in the 1960s and 1970s (McGinley, 2003). According to the SCRRA:

> "There was one incident at Mariposa Point (north of the pier) and one south of the pier. Both had service disruptions of about 24 hours. The failure mode was backwash: very large waves broke over the tracks and in running back to the sea carried the ballast and embankment. He said that he had seen pictures of the waves as high as the trains (that would be 15-16 feet above the rail). He said that the rip-rap there at that time was a smaller section than we have now."

This description of damages is consistent with severe wave runup and overtopping, and the resulting wave rundown removing armor stone from the protective revetment. It is noted that wave heights 15-16 feet above the rail are physically unlikely due to the spatial elevation of the revetment crest and railway. However, an observer standing on the railway might interpret overtopping spray in the presence of strong wind as wave heights. It is also noted that the rock protecting the railway during the observed incidents was smaller than the larger armor stone currently in place.

78. According to the SCRRA, there has never been a railway stoppage due specifically to conventional inundation. Conventional inundation, defined as still or standing water due to wave overtopping, has never caused a railway traffic service delay. This is intuitively expected as the railway is constructed on an elevated bed that greatly exceeds the elevation that can be reached by conventional still water coastal flooding.

79. There is no generally accepted guidance or methodology formulated to define the overtopping criteria necessary to interrupt railway service. Estimation of railway traffic service delays requires relating wave runup elevations to the physical positions (setback distances and elevations) of the SCRRA railway. Based on the above eyewitness account, criteria was established which combined runup elevation and a probability of occurrence. Runup elevation should equal or exceed 2 feet above the railway elevation, and thereafter there is a 50-50 probability that this overtopping condition would result in railway traffic service delay. Railway traffic service delay is treated equally and independently for each sub-reach.

3.1.3 Long Term Erosion / Recreation

80. Long-term shoreline erosional processes create damages through long-term profile translation and lost recreational opportunities as land is lost. Long-term beach erosion causes the mean shoreline position to translate landward. The landward advancing shoreline reduces the beach surface area available for recreation.

81. Long-term erosion is treated equally for each sub-reach. In each sub-reach, it is assumed that the existing shoreline will erode to the SCRRA railway, after which no further shoreline translation will occur. The railway will stabilize and fix the position of the shoreline, thereafter the long-term erosion rate no longer applies. Railroad structure damages due to direct loss of the structure from undermining are not considered.

3.2 Analysis Methodology

82. The integrated model developed for the present study combines the coastal engineering and economic sources of risk and uncertainty within a life-cycle framework. The life-cycle model generates a plausible storm condition, calculates various coastal engineering parameters, and determines shoreline erosion and other damage mechanisms. These are linked to the property inventory to estimate life-cycle property losses. The structure inventory used in this analysis and final economic results is fully developed and described in the Economics Appendix.

83. The general model flowchart is shown in Figure 12. Some of the significant model analysis concepts are further discussed below.

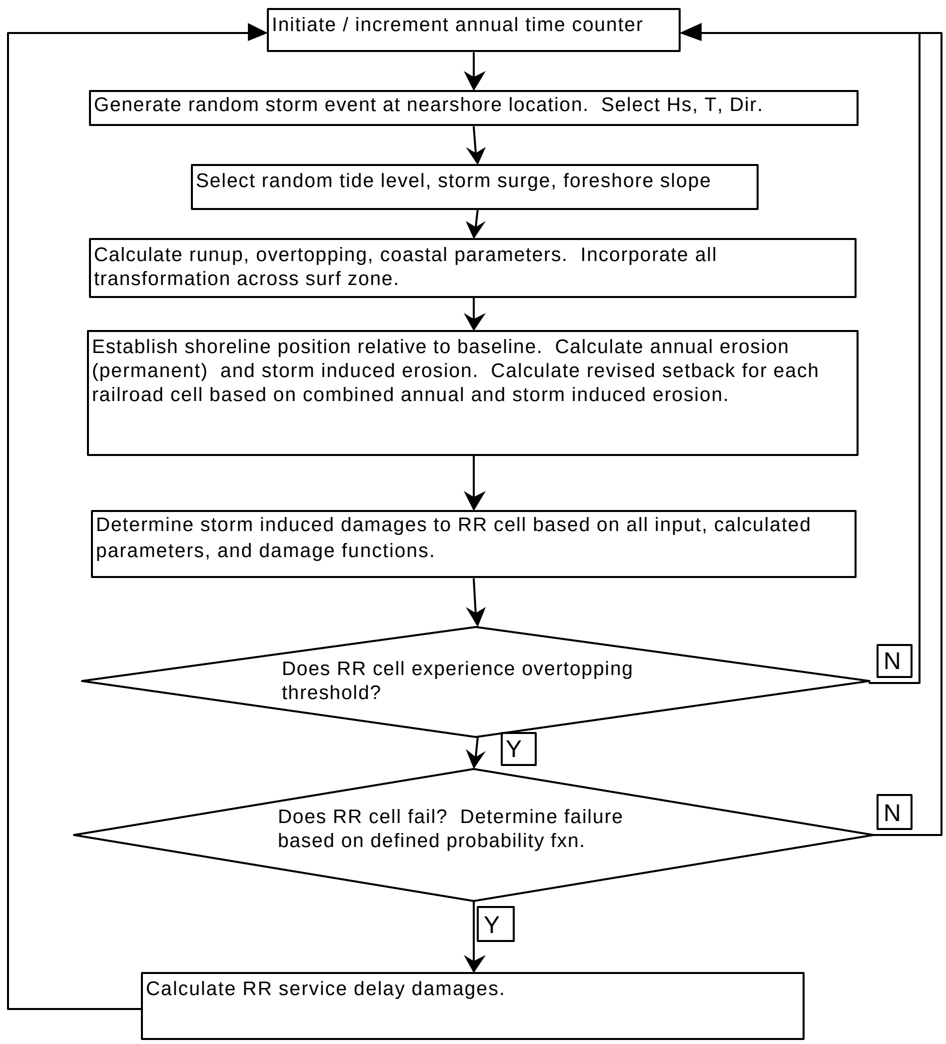

Figure 12 General Model Flowchart

84. Seasonality: This model is a single season model; seasonality of the wave climate is not considered. It is generally accepted that the most damaging storms in southern California occur during the winter months. Although it is recognized that very large wave events can and do occur during the summer season, the winter extratropical storms tend to cause the majority of the economic property losses. Inspection of the wave climate data indicates that the annual maximum wave heights occur in the winter months.

85. Single Storm: This model is a single storm event model; multiple storms are not considered. It is recognized that several independent storms occur every winter season. In fact, it is generally accepted that storm "clusters" are responsible for some of the most damaging storm years in southern California. This was clearly observed in the 1982-1983 storm season when a series of 6 discrete storm systems caused widespread coastal damages in southern California.

86. A single annualized storm is assumed to fairly represent the majority of the shoreline change and economic losses. It is well established that shoreline changes and the resultant economic damages tend to be cumulative throughout the storm season. However, there is little or no data that allows the full and quantitative delineation of the shoreline changes and economic damages attributed to individual storms in a cluster of successive storms. Shoreline change monitoring and measurement efforts typically are conducted on a time basis that is not in direct response to storm damages. Economic losses due to storm damages are fully described in the Economic Appendix. However, in general it is typical that detailed descriptions of economic damages are recorded and compiled well after the storm season has ended. Therefore, there is no reliable method to individually distribute and compartmentalize the combined shoreline changes and economic damages that do occur to a sequence of storms.

87. Each iteration of the Monte Carlo simulation assumes that the selected wave height is at least an annual storm event. Based on the annual maximum wave heights for the measured data record, the mean annual significant wave height is 2.60 m (8.5 ft). This value is consistent with conventional coastal engineering practice for expected annual maximums within the Los Angeles District.

88. <u>Independent Sediment Transport Processes</u>: Cross-shore sediment transport and longshore sediment transport were assumed separate and independent in this study. It is convenient to separate nearshore sediment movement into two components, longshore sediment transport and cross-shore sediment transport. This separation is not always valid in a strict sense because it is implicitly based on the assumption of plane and parallel profile contours. The time scale associated with storm-induced beach erosion is on the order of 1-3 days and depends on the level and duration of the wave and tide characteristics, whereas the time scale of beach adjustment is several months to years. Independence of cross-shore and longshore sediment transport relieves the responsibility of rigorously describing the joint morphological processes that occur. Independence of cross-shore and longshore sediment transport processes results in substantial numerical computational efficiency.

89. <u>Some Numerical Simulation Concepts</u>: A single annualized storm event provides substantial numerical simplicity over the numerical complexity inherent in a time-dependent simulation. A single annualized storm relieves the responsibility of rigorously describing the coastal and morphological processes that occur during a time-dependent sequence of storms.

90. The model employed in the present study is a spreadsheet based model in which the Monte Carlo simulation is conducted by a spreadsheet add-in package. Use of the spreadsheet allowed each year of the economic life-cycle to be represented by one column of the spreadsheet. Thus, the spreadsheet nature of the model eliminated the numerical requirement to iterate for each life-cycle year. Each iteration represented a new simulation that resulted in substantial numerical computational efficiency.

91. Simulation results were monitored for numerical convergence to evaluate the stability of output distributions during a simulation. A convergence criterion of 1.5% was established for convergence statistics for each output distribution.

3.3 Design Storm Water Levels

92. The design storm water level is derived from the linear superposition of the tidal range, an assumed range for storm surge, and sea level rise. It is recognized that tidal level and storm surge are physically independent and statistically independent. The design storm water level (without sea level rise) used in the present analysis ranges from +1.15-2.24 m (MLLW) (+3.8-7.4 ft). Design storm water level is determined based on the following expression:

Design Storm Water Level = TideUniform(0.85,1.64) + SurgeUniform(0.3,0.6) + SeaLevel(0.12*SimulationYear/50)

The tidal value used in the present analysis is randomly selected from a uniform distribution ranging between 0.85-1.64 m (2.8-5.4 ft). The surge value used in the present analysis is randomly selected from a uniform distribution ranging between 0.3-0.6 m (1-2 ft). It is assumed that the typical storm duration is three days and the typical storm duration peak is one day. The tide could be at any random position of the lunar cycle between neap and spring tides. During the one day storm period peak, the tide could range through the full unequal mixed range between neap tides and spring tides. It is assumed that the tide will rise to the highest tidal level. It is assumed that the neap tide high water condition will range up to the mean tide level, which is +0.85 m MLLW at La Jolla, CA. The mean higher high water condition is +1.64 m MLLW at La Jolla, CA. Thus, the tidal level was randomly selected from a uniform distribution ranging from 0.85-1.64 m MLLW representing the neap tide high and the spring tide high. It is assumed a constant storm surge occurs throughout the period of the storm and is assumed to occur simultaneously with the tidal level. Thus, the linear superposition of the random tide selection and the random surge selection minimizes a probabilistically unlikely set of tidal level / storm surge conditions, i.e. a 100 year recurrence tidal level occurring simultaneously with a 100 year recurrence storm surge level. The peak value of +2.24 m (7.4 ft) is slightly less than the highest observed water level (+2.38 m, +7.8 ft) that occurred during the 1983 storm season. The relative sea level rise was added at a linearly increasing rate between years 1-50 throughout the life-cycle simulation.

3.4 Design Waves

93. The design wave climatology used for this analysis is based on the winter wave climate previ ously discussed in Section 2.4. The significant wave height statistical analysis is shown in Figure 13. The lognormal distribution was selected as representative of the sample data set. The lognormal distribution has the form:

$$f(x) = \frac{1}{\sigma x\sqrt{2\pi}} \exp\left[-\frac{1}{2}\left(\frac{\ln x - \mu}{\sigma}\right)\right]^2$$

where μ = 0.74 (mean)

σ = 0.39 (standard deviation)

Although there is no theoretical basis for selection of the lognormal distribution, the lognormal distribution is commonly used in coastal engineering practice to represent significant wave height.

94. The lognormal probability distribution is truncated for significant wave height values below 2.60 meters (8.5 ft) to represent the annual maximum threshold. Each iteration of the Monte Carlo simulation assumes that the selected wave height is at least an annual storm event. Based on the annual maximum wave heights for the measured data record, the mean annual significant wave height is 2.60 m (8.5 ft). This value is consistent with conventional coastal engineering practice for expected annual maximums within the Los Angeles District. Thus, the simulation is forced to select a significant wave height from the input distribution greater than or equal to 2.60 m.

95. The lognormal probability distribution is truncated for significant wave height values above 8.0 meters (26.1 ft) due to wave breaking. Significant wave heights above approximately 8.0 meters will break. The concept of depth limited waves is well established in the literature. Based on empirical relationships derived by Weggel, a breaking height H_b can be computed for a given set of parameters including wave period, water depth, and local bottom slope. It can be shown that any wave with a height greater than this computed breaking height will also break. This is termed the depth limited height. For this analysis, significant wave heights in excess of 8.0 m at this 10.2 m depth are not considered due to breaking.

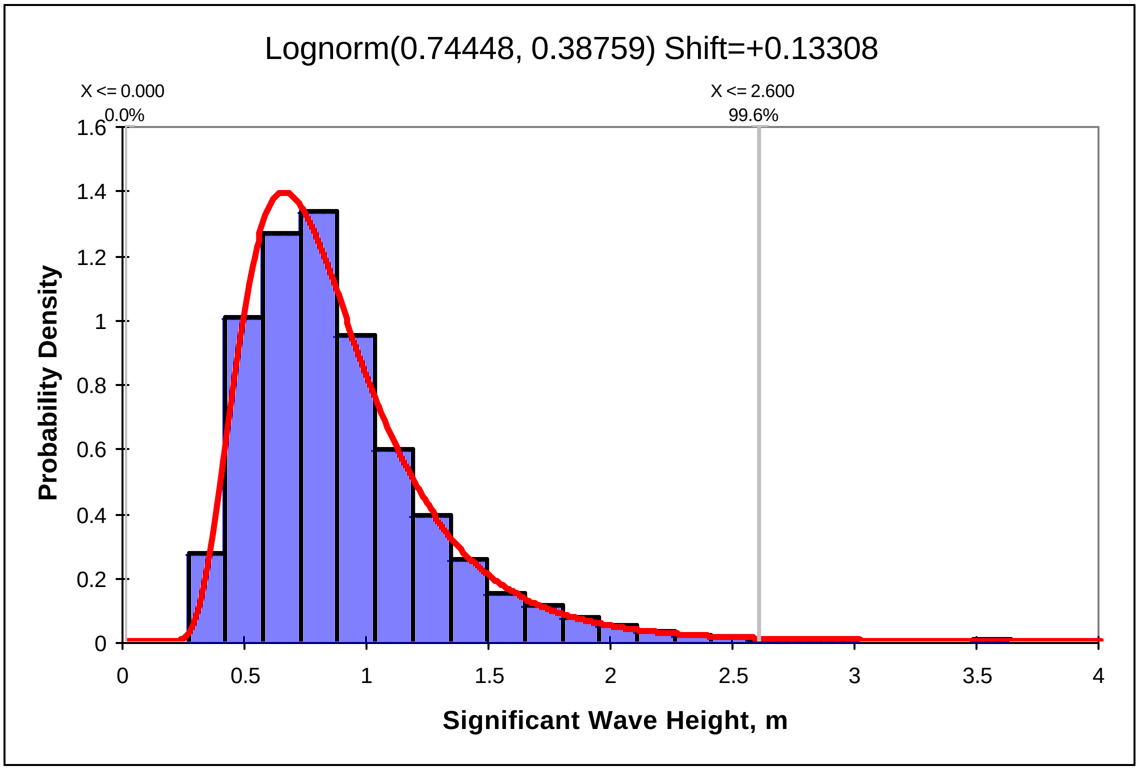

Figure 13 Probability Distribution for Significant Wave Height, Winter Data, 1983-1998

3.5 Design Wave Periods

96. The design wave period distribution used for this analysis is developed from the measured wave climate climatology previously discussed. The wave period statistical analysis is shown in Figure 14. The logistic distribution was selected as representative of the sample data set. The logistic distribution has the form:

$$f(x) = \frac{\operatorname{sech}^2\left(\frac{1}{2}\left(\frac{x-\alpha}{B}\right)\right)}{4B}$$

where

$\alpha = 13.2397$ (location parameter)

$B = 1.3463$ (scale parameter, $B > 0$)

$-\infty \le x \le +\infty$

97. There is no theoretical basis for selection of the logistic distribution, nor is the logistic distribution commonly used in coastal engineering practice to represent significant wave period. For this analysis the logistic distribution was selected for numerical convenience based on the best-fit distribution of the measured data.

98. The logistic probability distribution is truncated for values below 10 seconds and values above 20 seconds for purposes of the Monte Carlo simulation. The measured wave climate has been

demonstrated to have a dual sea/swell nature. It is common coastal engineering practice to identify 8-10 seconds as the delineation between sea and swell. For this analysis, 10 seconds was established as the lower limit for the wave climate. The logistic distribution was selected for purposes of numerical convenience and has a variate range of +∞. It is commonly accepted in coastal engineering practice that swell waves are typically not greater than 25 seconds. Inspection of the histogram for wave periods indicates essentially no waves greater than 20 seconds. For this analysis, 20 seconds was established as the upper limit for the wave climate.

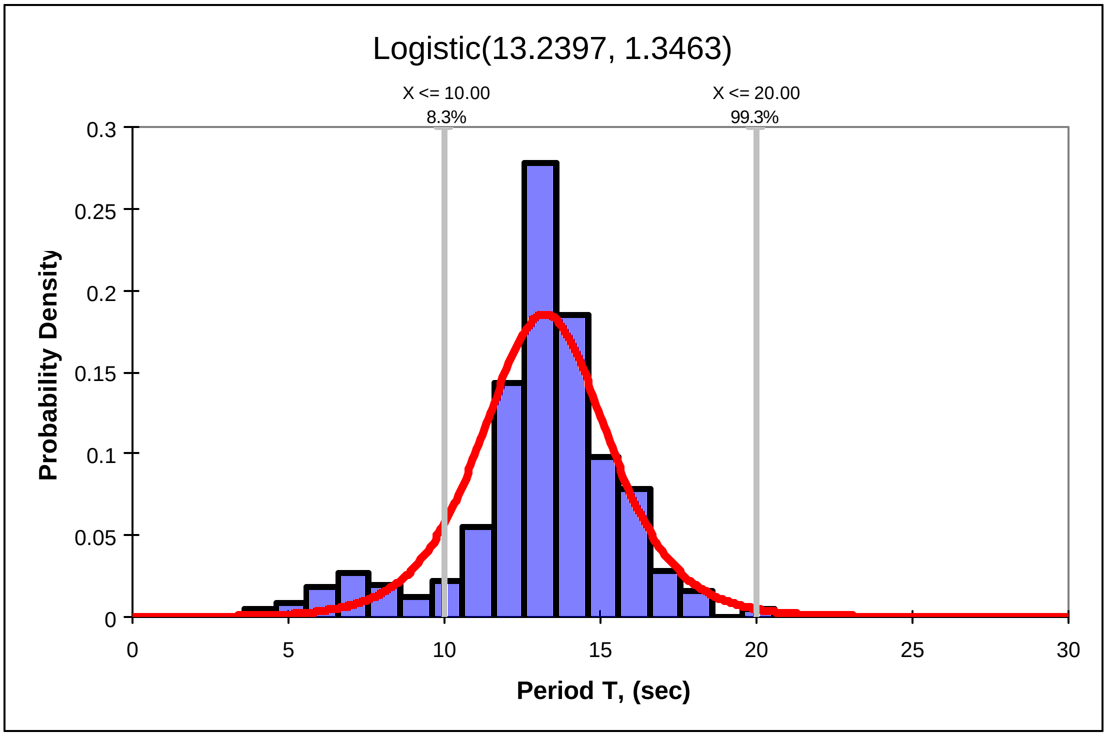

Figure 14 Probability Distribution for Spectral Peak Period, Winter Data, 1983-1998

3.6 Design Wave Runup

99. Runup is defined as the maximum vertical elevation of wave uprush above still-water level. Wave uprush consists of two components which include superelevation of the mean water level due to wave action (setup) and fluctuations about that mean (swash). In this analysis runup will be confined to the fluctuations about the mean where runup, R, is defined as a local maximum or peak in the instantaneous water elevation, 0, at the shoreline.

100. Wave runup is calculated independently for the shoreline conditions represented by the plane beach condition and the revetment condition. The runup is assumed constant within the entire reach since each reach is a uniform type of shoreline consisting of either plane beach or revetment. For each year of the lifecycle simulation runup was determined for each reach based on the input variables, e.g. wave height, wave period, beach slope. The calculated runup is added to the still water level for the final runup elevation.

101. The relative sea level change influences the calculation of runup. It was determined that relative sea level change will occur at a linearly increasing rate (Section 3.3). Increasing water depths allow higher runup values for the same incident wave heights. The still water level is incorporated indirectly into the

revetment runup calculation in the form of higher wave heights at the structure toe. Since runup is by definition the wave uprush elevation above still water level, the still water level is added directly to both shoreline conditions at the end of the calculation. Thus, an increasing trend is observed in the runup calculations.

Runup on Plane Beach

102. The method developed by Mase and detailed in the Coastal Engineering Manual (CEM) (USACE, 2002) was used. This predictive equation is valid for irregular wave runup on plane, impermeable beaches with slope ranging between 5H:1V – 30H:1V. The runup equation is empirically derived based on measured laboratory data and wave setup is incorporated into the calculating procedure. The runup equation has the form:

$$\frac{R_{1/3}}{H_0} = (0.75)1.38\zeta^{0.70}$$

where:

$R_{1/3}$ = the average of the highest 1/3 of the runups

H_0 = the significant deepwater wave height

ξ = surf similarity parameter calculated from deepwater wave height and length

103. Direct use of the runup equation developed by Mase results in runup values that are believed to be unrealistically large. This observation is consistent with discussion in the CEM where field measurements of runup are consistently lower than the predictive equations. There is some evidence that the runup equation overpredicts by a factor of 2 but is roughly the upper envelope of the data scatter. Therefore, a factor of 0.75 was applied to all runup calculations that more realistically reflects the runup experienced at San Clemente.

Runup on Revetment

104. The method developed by Hughes (Hughes, 2003b) was used in the present analysis. The general expression for irregular wave runup on smooth, impermeable slopes has the form:

$$\frac{R_{u2\%}}{h} = 1.75\left[1 + e^{-(0.47\cot\alpha)}\right]\left(\frac{M_F}{\rho g h^2}\right)^{\frac{1}{2}} \quad \text{for 1.5 <= cot " <= 4.0}$$

where:

$R_{u2\%}$ = runup elevation exceeded by 2% of the incident waves

h = water depth at structure toe

α = structure slope

M_F = wave momentum flux parameter (Hughes parameter)

ρ = fluid mass density

g = gravitational acceleration

The wave momentum flux parameter, M_F, is a nondimensional maximum depth-integrated wave momentum flux parameter (Hughes, 2003a). M_F will herein be referred to as the “Hughes parameter” for simplicity. The Hughes parameter characterizes the wave momentum flux, which is closely related to the force loading experienced by coastal structures or any other solid object placed in the wave field. An empirically derived expression for M_F has the form:

$$\left(\frac{M_F}{\rho g h^2}\right)_{max} = A_0 \left(\frac{h}{gT^2}\right)^{-A_1}$$

where:

$$A_0 = 0.639\left(\frac{H}{h}\right)^{2.026} \quad \text{and} \quad A_1 = 0.180\left(\frac{H}{h}\right)^{-0.391}$$

$H =$ significant wave height at structure toe

The calculated values of runup are further modified for surface roughness and other factors for consistency with the runup guidance detailed in the CEM which is based on the method developed by Battjes. The modification factors are given as:

$\gamma_r = 0.6$, roughness coefficient

$\gamma_b = 1$, berm coefficient

$\gamma_h = 1.17$, shallow water coefficient

$\gamma_\beta = 1$, wave angle of incidence coefficient

105. The following assumptions were made: significant wave runup value, impermeable slope, roughness consisting of 1 layer of rock, non-bermed profile, shallow water conditions (i.e non-Rayleigh distributed), and normally incident.

3.7 Design Foreshore Slopes

106. The design foreshore slope distribution used for this analysis is developed from the measured foreshore slope data previously discussed. The foreshore slope statistical analysis is shown in Figure 15. The extreme value distribution was selected as representative of the sample data set. The extreme value distribution has the form:

$$f(x) = \frac{1}{b}\left(\frac{1}{{}_e z + \exp(-z)}\right)$$

where

$$z \equiv \frac{(x-a)}{b}$$
$$a = 7.34 \text{ (location parameter)}$$
$$b = 1.94 \text{(scale parameter, } b > 0)$$
$$-\infty \leq x \leq +\infty$$

There is no theoretical basis for selection of the extreme value distribution, nor is the extreme value distribution commonly used in coastal engineering practice to represent foreshore slope. For this analysis the extreme value distribution was selected for numerical convenience based on the best-fit distribution of the measured data.

107. The extreme value probability distribution is truncated for values below 4 and values above 20 for purposes of the Monte Carlo simulation. The measured foreshore slopes has been demonstrated to have a range between 4 and 20. This range is typical of the foreshore slopes of other beaches located in Southern California.

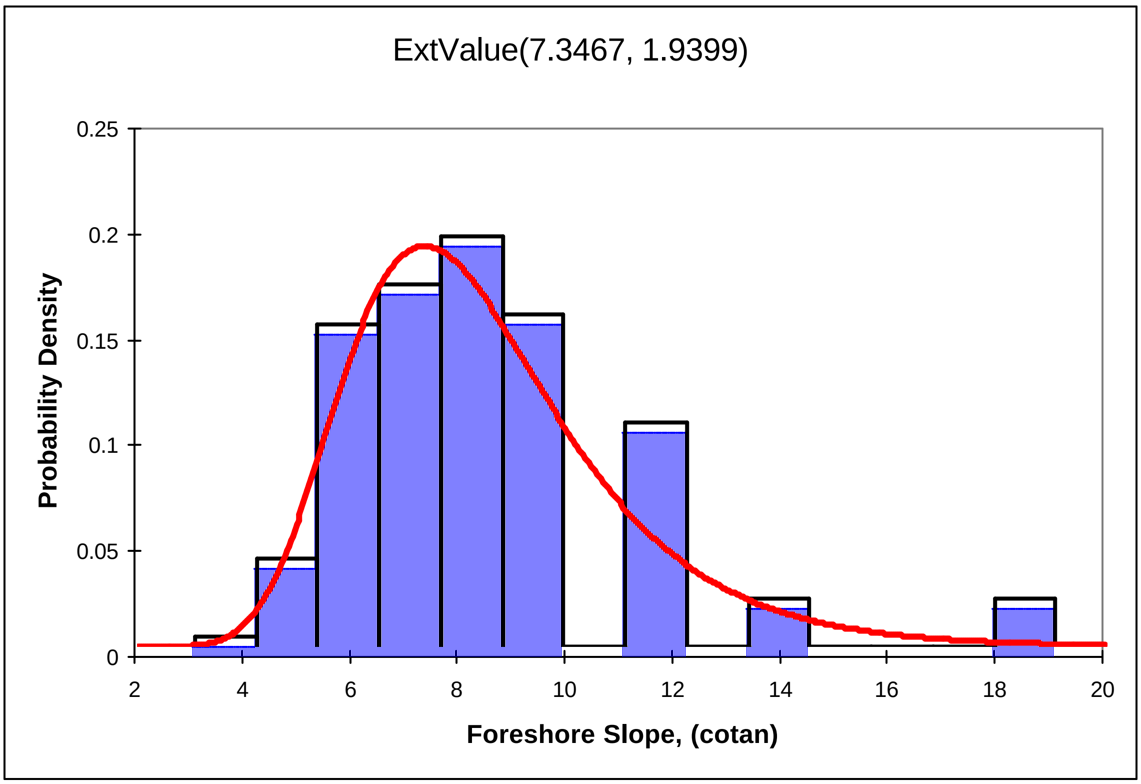

Figure 15 Probability Distribution for Foreshore Slopes

3.8 Design Beach Widths

108. Beach width in this study is defined as that portion of the beach between the foreshore berm contour and the backshore. This definition accounts for the “dry” portion of the beach. The foreshore berm contour was determined from a composite profile developed by overlaying multiple cross sections of

beach that was not fully eroded. Lines indicating the flat backshore and the sloped foreshore were intersected, yielding a +5.3 m (+17.4 ft) MLLW elevation representative of the foreshore berm.

109. The foreshore berm contour position was checked for seasonality by comparing the position of the currently measured contour, then making adjustments based on historical measurements. The 2002 LIDAR topographic survey clearly defined the location of the MHHW contour (+1.65 m). Recent repetitive beach profiles surveys have been conducted by the City of San Clemente (Coastal Frontiers Corporation, 2002) along historical Corps of Engineers survey transect locations. The survey data indicates the MHHW contour location fluctuates seasonally between 48.2-56.8 m (159-186 ft) for SC 1720 (Shorecliffs), 20.8-40.5 m (68-133 ft) for SC 1680 (Linda Lane), and 43.1-59.1 m (141-194) for SC 1623 (State Beach) respectively, from fixed transect backshore origins. The seasonal variations are 8.6 m (28 feet) for SC 1720, 19.7 m (65 feet) for SC 1680, and 16.0 m (52 ft) for SC 1623. The 2002 measured MHHW contour was measured in March, which is at the end of the winter season and thus the shoreline is in the most eroded condition. Therefore, the foreshore berm contour required no adjustment for seasonality.

3.9 Design Long Term Shoreline Change Rate

110. Long-term shoreline erosional processes create damages through long-term profile translation and the increasing potential for wave related damages. Long-term beach erosion causes the mean shoreline position to translate landward. The landward advancing shoreline reduces the beach width available for storm damage protection thereby increasing the probability of wave related damages to facilities and structures. As the shoreline retreats landward, the runup and overtopping damage zone increasingly encroaches on existing development. Landward encroachment of this wave-related damage zone will threaten existing development before the mean shoreline position could retreat far enough to directly threaten the structures by undermining. Long-term beach erosion also results in the gradual reduction of the beach surface area available for recreation.

111. The long-term shoreline change rates for the four locations that are historical to the CCSTWS were previously shown in Table 7 The shoreline change data sets are considered together with the sediment budget estimates to obtain erosion parameters representative for the entire study area. Based on these parameters a triangular distribution for long-term shoreline change rate was developed. The triangular distribution has a mean shoreline change rate of –0.10 m/yr (-0.33 ft/yr), the peak erosion rate is –0.21 m/yr (–0.7 ft/yr), the maximum erosion rate is –0.46 m/yr (-1.5 ft/yr), and the maximum accretion rate is +0.38 m/yr (+1.24 ft/yr). The distribution is shown in Figure 16.

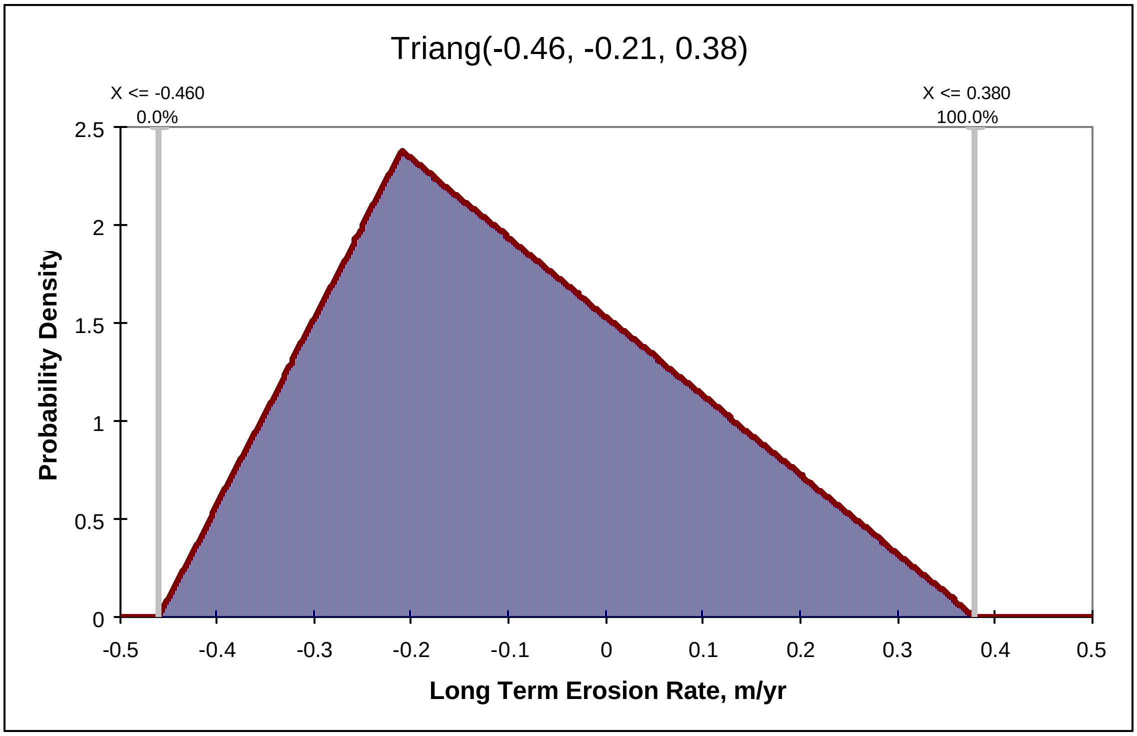

Figure 16 Probability Distribution for Long Term Erosion Rate

3.10 Design Storm Induced Beach Change

112. The design storm induced beach change used in this analysis is based on the storm induced beach change data previously discussed. The storm induced beach change statistical analysis is shown in Figure 17. The lognormal distribution, previously discussed in Section 3.4, was selected as representative of the sample data set. The following parameters were used:

μ = 3.01 (mean)

σ = 3.75 (standard deviation)

113. There is no theoretical basis for selection of the lognormal distribution, nor is the lognormal distribution commonly used in coastal engineering practice to represent storm induced beach change. For this analysis the lognormal distribution was selected for numerical convenience based on the best-fit distribution of the measured data.

114. Storm-induced erosion effects on the profile are superimposed on the present and future shoreline positions. Storm profile response can be considered directly correlated to storm severity and therefore significant wave height. Storm induced shoreline response is used to quantify the land loss and associated structure loss during the Monte Carlo simulation. The simulation computes the land loss and structure damages due to storm-induced profile change at annual intervals through the 50 year life cycle. The simulation randomly samples a storm event, and allows the shoreline to retreat according to the amount associated with the storm. If a structure is located within this storm induced retreat distance, the structure is considered lost and removed from the inventory. The shoreline position is reset to the pre-storm position to avoid double counting of erosion. It is idealistically assumed that the storm-induced profile change is a temporary event and the beach will recover to its pre-storm position. Thus, an artificially induced double counting of the storm-induced erosion is avoided.

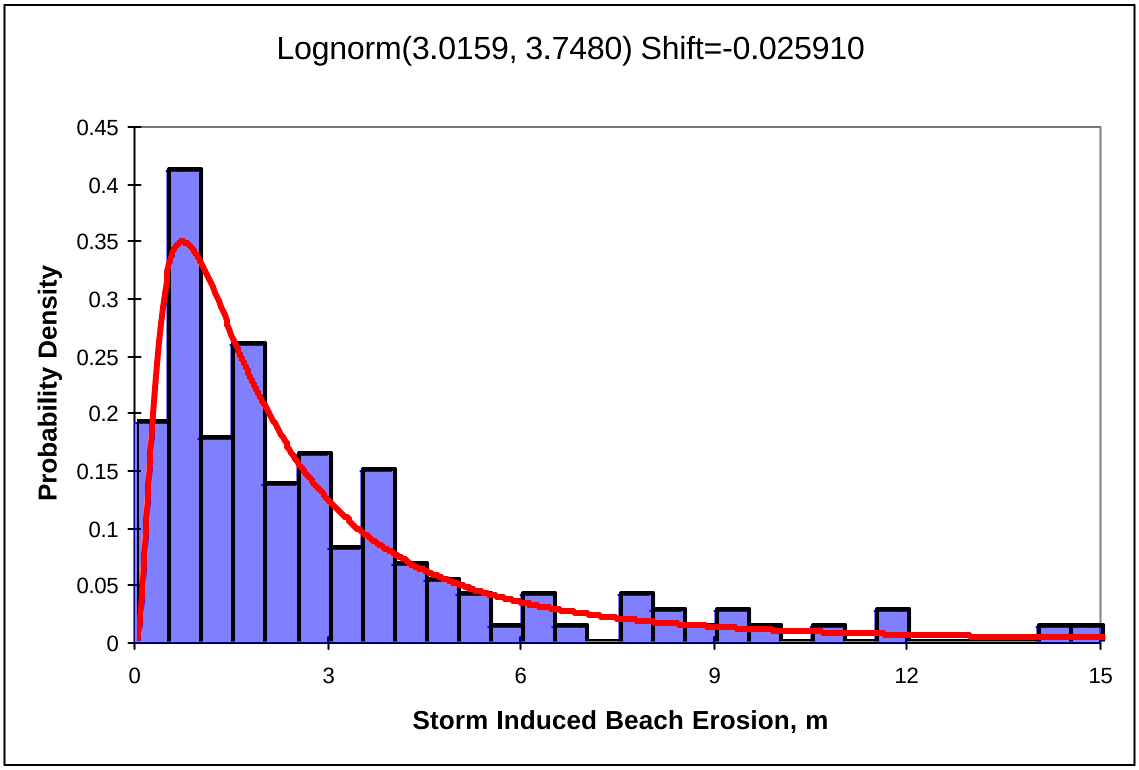

Figure 17 Probability Distribution for Storm Induced Beach Change

4 WITHOUT-PROJECT CONDITIONS

4.1 SCRRA Track Maintenance Operations

115. The following description and account of maintenance activities was provided by the SCRRA (McGinley, 1998). The detailed account of railway activities is excerpted to highlight those activities that are directly pertinent to the analysis methodology conducted in this study.

> "Under 49 CFR 213.103, the ballast supporting the track must be constructed and maintained to support the track (and trains) and restrain the track from lateral, longitudinal, and vertical movement. In order to maintain this ballast material, it must be protected against erosion by ocean waves. These ocean waves have two effects, they can undercut the embankment, causing the ballast to drop from the track area, and they can dislodge the ballast through their impacts and the resulting backwash. …"

> "The method used to resist this erosion for the last several decades has been to place broken stone rip-rap along the ocean side of the track embankment. (In earlier times, some rip-rap was waste concrete, however that material tended to break up leaving exposed reinforcing steel exposed to persons on the beach.) The size of the stone is from four feet in maximum dimension down to four inches. The stone has come from various quarries in Southern California. Almost all of the rock applied in the 1990 is granite that has come from Corona or Newberry Springs. The rock is delivered to the roadbed area in railroad cars which dump to the side. Before and after dumping the rock, construction equipment (tracked excavators and rubber tired end loaders) is used to arrange the rock into a uniform row, and to keep it clear of the PUC walkway area alongside the track. This work is done at night because of the need to occupy the tracks with the

rail cars. The rock is placed on a 1:1 or 1.5:1 (horizontal to vertical) slope towards the beach, and to a height of about three or four feet higher than the rails. This height is to deflect large waves at high tides, which are otherwise observed to impact the tracks. This placement and replacement of rip-rap is all fixed in location to the 20 to 30 feet west of the centerline of the tracks. … The SCRRA experience since 1993, and staff experience dating into the 1970's, is that the very steep piles of rock, often underlain by compacted sand. The next stage of erosion is that the high parts of the piled stone settles into these voids, lowering the height of the embankment. This is an approximately annual cycle, with the most erosion occurring in the winters. … Over the last five years, rock has been installed about annually to replace that lost to erosion."

"The use of random-dumped stone rip-rap has evolved over decades of practical experience and has not been studied or engineered by rigorous methods. It is believed to present an exposure to the ocean which is able to absorb significant energy and to capture some of the smaller stone and sand. …A more refined barrier against erosion such as large stone set into a planned matrix or a cast concrete wall is too expensive to install under regular railroad maintenance budgets and simply has not been considered. It would appear that a more elaborate barrier would disrupt the beach during construction."

4.2 SCRRA Construction and Operations and Maintenance Costs

116. Estimation of construction and Operations and Maintenance (O&M) costs associated with the revetments is required for consideration in the economic analysis. Costs are quantified for existing O&M practices, future construction projects, and O&M of future construction projects. Structure construction and O&M costs are not considered north of the Capistrano Shores development or south of San Mateo Point.

117. Existing O&M cost information has been provided by the SCRRA. These O&M costs are those directly attributable to the revetments that were constructed in response to direct wave related damages, and are separate from those costs incurred during routine O&M of the railroad. The revetments have suffered documented damages over its service life. The revetments were constructed incrementally at various times since at least the 1930's according to SCRRA information sources; various levels of engineering and design (or none) were employed. Over the ensuing years the structures have been subjected to several storms of varying intensities. Degradation of the revetments has occurred as evidenced by many instances of over-steepened slopes and ejected and/or displaced armor stones that reduce the overall structural integrity. This has led the SCRRA to conduct maintenance operations on the revetments at various times throughout the revetment life-cycle. Information provided by the SCRRA indicates that approximately $300,000 every 3 years have been expended on these maintenance operations; this equates to approximately $100,000 per year. The SCRRA was unable to provide detailed information that would allow determination of exactly where along the railroad the O&M effort was applied over the years. Therefore, this $100,000 per year average O&M cost was distributed on a per meter basis throughout the four reaches that are currently protected by revetment.

118. Future damages are expected during the without-project condition life cycle. Future damages are expected to increase due to an increase in the length of railroad exposed to the ocean as the beach erodes. Therefore, future costs are expected to be greater than the current rate of expenditures. These future costs were estimated for various scenarios. It is noted that Reach 2, Reach 4, Reach 6, and Reach 8 are the original ballast construction. Reach 1, Reach 3, Reach 5, and Reach 7 are the improved revetment reaches.

119. Three types of construction and their resultant O&M costs were developed and are shown in Table 11. The construction first costs were based on similar structures built recently in the southern California area. It usual and customary to estimate O&M costs equal to approximately 1% of the construction first cost. The three types of construction are:

a. Existing Condition: The SCRRA is currently placing large armor stone via side dump methods as previously described. This was deemed a "non-engineered" revetment. The $100,000 per year average O&M cost incurred under existing conditions was distributed on a per meter basis throughout the four reaches that are currently protected by revetment.

b. Engineered Revetment: It may become necessary or mandatory for the SCRRA to construct "engineered" revetments. This would entail utilizing a detailed engineering and design process resulting in plans and specifications, and placing the armor stone under controlled stone placement methods. An engineered revetment would provide increased hydraulic stability over current non-engineered methods. It was assumed that an engineered revetment would largely utilize many of the same revetment parameters as is currently used in the non-engineered placement methods. This would include: revetment elevation, revetment, slope, armor stone size and gradation, armor stone purchase unit price. The significant difference is the development of the engineering and design effort and the controlled armor stone placement methods. The engineered revetment was subdivided into two types:

1) Construct engineered revetment on existing ballast railroad.
2) Upgrade existing non-engineered revetment to an engineered revetment.

c. Seawall: It may become necessary or mandatory for the SCRRA to construct seawalls. The seawall was assumed to be steel sheet pile with concrete cap, common in the southern California area. This would entail utilizing a detailed engineering and design process resulting in plans and specifications. A seawall would provide increased hydraulic stability over current non-engineered methods.

Table 11 Construction and O&M Unit Prices

	First Cost, $/m ($/ft)	O&M Cost, $/m ($/ft)
Existing Condition		$43.46 ($13.25)
Revetment 1 (ballast)	$1968 ($600)	$19.68 ($6.00)
Revetment 2 (upgrade)	$656 ($200)	$19.68 ($6.00)
Seawall	$9840 ($3000)	$98.40 ($30.00)

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